DE PONTIBUS:

A POCKET-BOOK

FOR

BRIDGE ENGINEERS.

BY

J. A. L. WADDELL,

C.E., B.A.Sc., Ma.E.;

Knight Commander of the Japanese Order of the Rising Sun; Consulting Engineer, Kansas City, Mo.; Member of the American Society of Civil Engineers; of La Société des Ingénieurs Civils, Paris; of the Rensselaer Society of Engineers and the Society for the Promotion of Engineering Education; and Honorary Member of the Kogaku Kyokai (Japanese Engineering Society.)

FIRST EDITION.

FIRST THOUSAND.

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1898.

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To

McGill University,

the representative institution of learning of the
Dominion of Canada
(the author's native country),
this little treatise is

DEDICATED

as a mark of the author's grateful appreciation
of the distinction accorded him
by that university
in 1882,
in conferring upon him two engineering degrees.

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PREFACE.

In presenting to the public a new technical work, it is the custom to offer some sort of apology for its appearance ; hence the author of this treatise, in order not to be considered peculiar, feels it incumbent upon him to do likewise. Moreover, there is in this case a good reason for beginning his book with an apology, because of his audacity in imposing upon the good nature of the engineering profession by asking its members to read still another work upon the already overwritten subject of bridges. If he could do so, the author would here plead *primum tempus;* but this is by no means his first offence. Perhaps, though, the fact that it is twelve years since the appearance of his last book (pamphlets, of course, excepted) will be considered by his critics as a "mitigating circumstance" in this case.

But, to speak seriously, if this work were a mere rehash of other books, or if it dealt with the same old, worn-out subjects, the author would not presume to present it to the engineering profession ; but, on the contrary, in writing it he has endeavored to make the contents as original as possible, and to treat essentially of the fundamental principles of bridge-designing and their application. It will be noticed throughout the book that quotations from other works on bridges are "conspicuous by their absence," and that the author has drawn almost entirely upon his own professional practice for examples to illustrate his text. For the latter no apology is required, because his own designs (as far as the process of development has permitted) have naturally been made in conformity with the principles which he herein offers as a guide to bridge-designing ; and they are therefore more appropriate as illustrations than the designs of others.

The author desires it to be distinctly understood at the outset that he by no means claims that the methods of designing and construction given herein are all either original with him or are the only correct methods; they are, however, the very best of which he knows, whether they originated in his practice or were adopted in whole or in part from the practice of others.

Probably the first point in connection with this book which each reader will find to criticise is its peculiar title. Each will probably remark, "Why, in the name of common sense, did the author choose such an indefinite and outlandish title as 'De Pontibus'?" Reader, its indefiniteness is its most praiseworthy feature; for the work is certainly not a complete treatise on bridges, being eminently lacking in illustrations of details, and entirely without any treatment of the theory of stresses; and what title could be more appropriate to such a book than the indefinite one, "Concerning Bridges"? But, the captious reader will reply, "Why revert to the Latin language? Is not English good enough?" Certainly it is; but the author had a sound reason for using the Latin, which he will proceed to explain, as the said captious reader will assuredly not be satisfied without some explanation.

For five consecutive years of his early life the author devoted more than half of his working time to the study of the Latin language; and this is the first opportunity which has occurred during the twenty-two years of his professional career to put the knowledge (?) so obtained to any practical use. Moreover, he fears that, even if he be so fortunate as to be able to practise his profession another twenty-two years, no other occasion will occur to use it, so he feels the necessity for grasping this unique opportunity of a lifetime.

Captious reader, are you now satisfied?

That many readers will have faults to find with the book goes without saying. Some may object to its incompleteness, in that it does not treat of stresses and that it gives principles without actual examples of their application. To these the author would reply that any information desired concerning the subject of stresses can be found in such standard works on bridges as those of Profs. Burr, Du Bois, and Johnson, and that, if he were to attempt to illustrate the principles by actual examples of designing, his book would never be finished.

As stated in Chapters XI and XIX, the second edition of the author's "General Specifications for Highway Bridges of Iron and Steel" and the first edition of his "Compromise Standard System of Live Loads for Railway Bridges and the Equivalents for Same" are now exhausted, and will not be reprinted, as this treatise will replace them.

In writing Chapters XV, XVI, XVII, and XVIII it was found necessary to copy certain portions of Chapter XIV in order to make the various specifications complete; but the amount of repetition was made as small as possible by referring, wherever no changes were introduced, whole sections of one set of specifications to the corresponding sections in a preceding set.

The subject of suspension bridges is not dealt with in this work, partly because until lately the author has not paid much attention to this class of structures, and partly because they are so different from other bridges, being suitable for very long spans only, that each suspension bridge requires special specifications of its own.

The author has the presumption to hope that there will be considerable demand for this book, for he considers that it will be useful to the following classes of readers: first, to practising bridge-engineers, because of many little suggestions that will help them to effect improvements and to avoid mistakes; second, to young engineers in offices of bridge specialists and of bridge-manufacturing companies, for perfecting them in their work; third, to professors of civil engineering, to show them the practical side of bridge-designing and building, and to aid them in giving their lectures on bridges; fourth, to students of civil engineering, as a supplementary text-book that will enable them to understand the application of what they have learned during their course in bridges; fifth, to railroad engineers, because of the bridge specifications contained, and to instil into their minds the importance of having their bridges properly designed, manufactured, inspected, shipped, and erected; and, sixth, to a few county commissioners, who may desire to obtain through the specifications good highway bridges at minimum legitimate cost.

The author has endeavored to make the various specifications in this book thorough, correct, and complete. If he has failed to do so in any particular, he would feel deeply indebted to any one who will point out to him how and where; and he would be grateful to any reader who will inform him of any typographical or other errors that he may discover; for all errors found in the first edition will be corrected in the second, provided the work be well enough received by the profession to warrant the issue of another edition.

In conclusion the author desires to acknowledge with many thanks his indebtedness to his assistant engineers, Ira G. Hedrick, Assoc. M. Am. Soc. C. E.; Lee Treadwell, M. Am. Soc. C. E.; and John L. Harrington, Jun. Am. Soc. C. E., for valuable aid rendered him in the preparation and checking of the MS. of this work.

KANSAS CITY, MO., Oct. 18, 1897.

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DE PONTIBUS.

CHAPTER I.

INTRODUCTION.

WHILE it is true that the development of bridge-building in America owes much to the system so long in vogue of bidding on competitive plans, in that such competition has tended to sharpen the wits of the engineers of the competing companies, it is equally true that the said competition has done all the good it can for the science of bridge-designing, and now acts as a clog to prevent its further advancement. The correctness of this assertion scarcely needs any demonstration, but it may be well, notwithstanding, to give here a few reasons therefor.

As human nature is the same the world over, and as men in general are working for the almighty dollar, it stands to reason that when a bridge-company's engineer is preparing upon fixed specifications a design to be used as a basis for a competitive bid, and when he knows that in nineteen cases out of twenty the contract will be awarded to the lowest bidder whose design conforms to the letter of the specifications, although it may not be up to the requirements of good engineering practice, he will take advantage of every weak point and omission in said specifications, even if his engineer's conscience proclaim the design he submits to be worse than faulty. As it is entirely practicable to take advantage of any set of railroad-bridge specifications yet published, it is evident that as long as competitive, lump-sum bids are the fashion, just so long will railroad bridges be badly designed. As for highway bridges, their letting, designing, and construction are so often left in the hands of such incompetent and unscrupulous parties that, until some fundamental change in existing conditions be effected, nothing can be done to improve the present unscientific, wretched, and even criminal methods of highway-bridge building.

Concerning the prejudicial effect of competitive designing upon the development of the science of bridge-engineering, the author can speak authoritatively, because for about six years he acted as engineer to a bridge company. During that time he lost many contracts for small bridges because he insisted on incorporating in his designs certain features requiring extra metal, which features he considered essential, although they were not called for in the specifications. On the other hand, he once earned a commission of more than ten thousand dollars upon a single piece of work by knowing how to take the greatest advantage of the specifications upon which bids were requested. In defence of this action, however, it must be mentioned that it was understood at the outset that, after the selection of the successful competitor, the contract was to be adjusted upon the basis of a pound price for the metal-work. By reason of this feature, the author was able to correct later on all the weak points of his preliminary design to such an extent that the structure until within a few years was by far the best of its kind built.

This case is given merely as an illustration of how great are the possibilities for trimming a design which is based upon ordinary standard specifications, and how great is the temptation to take advantage thereof. Another way to illustrate this point is to compare the weights of the structures manufactured and built by any bridge company by the lump sum with the weights of similar structures manufactured and built by the same company for a pound price. The difference in weight often runs as high as fifteen or twenty per cent or even higher, if there be no supervising engineer to hold the bridge company in check.

For a railway company, the most satisfactory method of building bridges is either to have a permanent, competent bridge engineer in its employ, or to retain some specialist of established reputation to prepare specifications and complete detailed plans (not working drawings, however) for all its bridges, and to provide competent inspectors to see that during manufacture, shipment, and erection the plans and specifications are strictly followed.

The necessity for the specialist to stand between the purchaser and the manufacturer of structural steel is, as a general rule, not appreciated by the purchaser, unless he has already had some experience in letting contracts for and in the building of steel structures without engineering aid in the designing and supervision. When the purchaser puts himself in the hands of the manufacturer, he is pretty sure to get the worst of it; for if the contract be let by schedule prices the structure is liable to be loaded down with useless metal, while if the contract be let for a lump sum the structure will probably be ruined by having the metal "skinned" out of it, especially in the most important parts, viz., the details. Moreover, the manufacturer is seldom capable of evolving a truly first-class design, for the reason that his training has always been in the line of his own pecuniary interests, which are to obtain the maximum of pay for the minimum of structure; so that, even when given all the metal and money that he could ask for when preparing a design, he would not succeed in making a really good one, simply because of not knowing how.

On the other hand, the specialist should stand between the contractor and the purchaser, so as to see that the latter does not take any undue advantage of the former by means of a harsh or unjust interpretation of the specifications, especially when the contractor has suffered loss or delay on account of causes beyond his control.

Occasionally a manufacturer offers to prepare the plans for certain portions of the work on the plea that he has had so much more experience in such matters than the engineer. The acceptance of this offer by either the purchaser or the engineer is a mistake; for the engineer, if he have sufficient ability to warrant his being retained on the work, can by careful study almost always evolve a better design than can the contractor, even if it be the first experience which the former has had in connection with the portion of the work under consideration. On two or three occasions only, and several years ago, the author was either induced or compelled by the purchaser to defer to the greater experience of the contracting engineer; and in each case he has had reason to regret the concession; so he has concluded that in future he will receive with thanks any suggestions which the manufacturer may offer, give them due consideration, and then make the design as he himself sees fit.

Considerable opposition to the methods of design advanced in this work and to the specifications given is anticipated, on the plea that the requirements are too exacting and that the class of work called for is unnecessarily refined and consequently expensive. To such opposition the author would reply as follows: First, the designing and building of bridges and similar structures cannot be too well or carefully done; and, second, that within the last three years, upon some fifty or sixty thousand tons of the author's work designed in accordance with the said methods and built in accordance with the said specifications, the prices quoted by the competing manufacturing companies were extraordinarily low, and that no complaint of any account has since been raised by the manufacturers in respect to the expense involved by either the designs or the specifications.

The principles of design given in the succeeding chapter should be adhered to in all structural metal-work; and any violation of any one of them is a mistake that will be regretted sooner or later by the parties owning the structure. Many of these principles are violated constantly by shop draftsmen, even when the engineer's drawings show the details correctly. This is due partly to custom in designing certain details in certain ways, and partly to the ignorance of the draftsmen. The author would urge upon young engineers who are working on plans for structural metal-work to adhere to the principles herein given whenever it is practicable for them to do so. Had more attention been paid to first principles of design when the plans for most of the New York and Brooklyn elevated railroads were being prepared, millions of dollars would have been saved. This statement can be verified by a perusal of the author's paper on Elevated Railroads, referred to and quoted from in Chapter VIII, more especially the *résumé* of the discussions and Mr. Hedrick's report on the said New York and Brooklyn elevated railroads.

In spite of all the care that the most expert designer can give to his work, errors of greater or less magnitude will creep in occasionally; and one can always improve somewhat upon any finished design. Such being the case, it follows that the designing of steel structures should be intrusted to expert and disinterested designers only, instead of, as is generally the case, to the cheap draftsmen employed by the manufacturing companies.

There are a few features of the specifications given in Chapters XIV. and XVIII. which will require a little explanation or comment. This will be given in this chapter, the various items being treated, as nearly as may be, in the order in which they occur in the said specifications.

"A" Truss Bridges.

This style of structure, originated by the author and covered by letters patent, is a four-panel truss-bridge having eye-bars in bottom chords and centre verticals, and rigid members for all the other portions of the trusses and for the entire lateral system. It was evolved in this way: For a number of years the author was dissatisfied with all railroad bridges for spans between the superior limit of the plate-girder and a length of about one hundred and fifty feet, ordinary pin-connected, through, Pratt trusses being too light and vibratory, and the riveted bridges as then built being clumsy, unscientific, and uneconomical. On this account he tried for some time to find an opportunity to experiment upon a design of his own to fill a portion of the gap, but the opportunity did not occur until April 1893, when he was retained by the General Manager of the Kansas City, Pittsburg, and Gulf Railroad Company to design some bridges. After a little persuasion the General Manager was induced to agree to build a 100-ft. "A" truss span as an experiment; but when he saw the completed plans he ordered at once four bridges to be built therefrom, and this style of structure was soon afterwards adopted as the standard 100-ft. span for the road.

These bridges have shown such rigidity under traffic that they have been used on the St. Louis Southwestern Railway, and have been adopted as the standard for spans between sixty-five feet and one hundred and sixteen feet by the Nippon Railway Company of Japan.

The advantages of this type of bridge are great rigidity in all directions, ease and cheapness of erection, and economy of metal when it is compared with structures of other types having equal strength and rigidity.

IMPACT.

The uncertainty as to the magnitude of the effect of impact on bridges has for many years been a stumbling-block in the path of systemization of bridge-designing, and will continue to be so until some one makes an exhaustive series of experiments upon the actual intensities of working stresses on all main members of modern bridges of the various types. The making of these experiments has long been a dream of the author's, and it now looks as if it would amount to more than a mere dream; for the reason that the general manager of one of the principal Western railroads has agreed to join the author in the making of a number of such experiments on certain bridges of the author's designing, the railroad company to furnish the train and all facilities, and the general manager and the author to provide the apparatus and experimenters. It is only lack of time that has prevented these experiments from being made this year, and it is expected that they will be finished in 1898. It is hoped that the result of the experiments will be either to determine a proper formula or curve of percentages of impact for railroad bridges, or else to inaugurate a series of further experiments that will determine it.

Meanwhile the author has adopted temporarily the formula given in Chapter XIV., viz.,

$$I = \frac{40000}{L + 500},$$

in which I is the percentage for impact to be added to the live load, and L is the length in feet of span or portion of span that is covered by the said load.

This formula was established to suit the average practice of half a dozen of the leading bridge engineers of the United States, as given in their standard specifications, and not because the author considers that it will give truly correct percentages for impact.

In spite of all that has been said to the contrary in the past or that may be said in the future, the impact method of proportioning bridges is the only rational and scientifically practical method of designing, even if the amounts of impact assumed be not absolutely correct; for the method carries the effect of impact into every detail and group of rivets, instead of merely affecting the sections of the main members, as do the other methods in common use.

The assumption made in some specifications that the live load is always twice as important and destructive as the dead load, irrespective of whether the member considered be a panel suspender or a bottom chord-bar in a five-hundred-foot span, is absurd, and involves far greater errors than those that would be caused by any incorrectness in the assumed impact formula.

The author acknowledges that he anticipates finding the values given by the formula somewhat high; but it must be remembered that the said formula is intended to cover in a general way, also, the effects of small variations from correctness in shop-work, or to provide for what the noted bridge engineer, the late C. Shaler Smith, used to term the factor of ignorance.

Using a uniform tension intensity of 18,000 lbs. for eye-bars and 16,000 lbs. for built members will strain the metal up to nearly one half of the elastic limit shown by specimen tests, and probably somewhat higher than one half of same shown by tests of full-size members. So long as the greatest actual intensity of working stress is kept in the neighborhood of one half of the elastic limit, sufficient precaution has been taken against all possibility of failure by load even in the far-distant future.

The impact formula for highway bridges given in Chapter XVI., viz.,

$$I = \frac{10000}{L + 150},$$

was established to fit the author's practice. Its correctness is not likely to be ever determined by experiment.

MEDIUM STEEL.

The reason for using this metal almost exclusively and barring out soft steel, except for rivets and adjustable members, is because the two kinds of the raw material cost almost exactly the same per pound, while medium steel is the stronger of the two by fully fifteen per cent, and is in every particular just as reliable and satisfactory for use as soft steel. The only advantage claimed for the latter is that it requires no reaming, which, in the author's opinion, is a fallacy, because, in order to obtain proper matching of rivet-holes in the various component parts of a piece, reaming is a *sine qua non.*

HIGH STEEL.

The use of this material is limited to those portions of very long spans where the impact is small, and where there is neither reversion of stress nor any condition even approaching same. The specifications bar out its employment for intermediate posts of simple trusses, because in modern, long-span bridges the top chords are so curved that the stresses in vertical posts either reverse or vary between wide limits.

SUBPUNCHING AND REAMING.

To inaugurate the exclusive use of this process for all important work in structural steel, the author has fought a long and bitter fight with the manufacturers, and it begins to look as if he were going to win eventually. At any rate he has succeeded in having it adopted on all of his own work for several years past in spite of a most powerful adverse influence brought to bear upon the purchasers by certain of the largest manufacturers in the United States. Again, in his paper on Elevated Railroads he has advocated most unequivocally the adoption of subpunching and reaming on all important metal-work, and his views have been indorsed by a majority of the engineers who discussed the subject. Any reader who is in doubt concerning this question is advised to read all that is written on the subject in the original paper, the discussions, and the *résumé*, all of which have been published in the *Transactions* of the American Society of Civil Engineers for 1897.

PAINTING.

In respect to painting structural steel, engineers as a body appear to be unsettled in opinion. Each one either has a pet paint of his own or else is experimenting in a haphazard way to find one. In the *résumé* of the aforesaid paper on Elevated Railroads the author wrote as follows in respect to this matter:

"In short, the engineering profession is all at sea on the paint question, and is likely to remain there until there is some organized investigation made. In the author's opinion the subject is one of sufficient importance to warrant the appointment of a special committee of the Society to experiment and investigate on the subject for a term of years until some valid conclusions can be reached."

A short time ago the author followed this with a formal proposition to the Society to appoint such a committee; but the motion after considerable discussion both *pro* and *con* was lost. Considering the fact that the people of the United States are investing annually many millions of dollars in structural steel, and that no satisfactory preservative for the metal has yet been found, one would think that the question of the best kind of paint for metal-work and the best method of painting would be a proper subject of investigation for a special committee of the American Society of Civil Engineers.

Since this negative vote was cast, the technical papers have stated that certain parties in New York City have at considerable expense inaugurated a series of practical tests of a number of brands of metal-work paint. The results thereof ought to be of great value to the engineering profession; but the good work should be carried still farther by an authorized body of well-known engineers, who would be willing to devote a portion of their time for many years to the investigation.

A perusal of this introductory chapter may cause the reader to think that the author is at variance with the manufacturers of structural steel; but such is by no means the case, for his relations with them on construction are almost invariably of the most pleasant description. Moreover, the assertions made herein concerning the opposition of steel manufacturers in general to improvements in design and in the quality of the manufactured product do not apply to all of the manufacturers of structural steel in the United States; because there are several companies who are always ready to do anything in reason to aid an engineer in making investigations and improvements, and who are continually putting in new machinery for the purpose of bettering their output. It is altogether natural that the manufacturer should try to make all the money he can by adopting simple details which are easily manufactured and easily put together in the field, and by avoiding such slow and tedious shop processes as subpunching and reaming; and it is also altogether natural that the consulting engineer should use every endeavor to secure certain results in finished structures which are in the line of improvement and of ultimate economy for his employers: hence it is to be expected that the said manufacturers and engineers will occasionally disagree, and that each party will battle for his supposed rights. This is, however, no reason for ill feeling or for any conflict between the manufacturer and the engineer after the contract is awarded on fixed plans and specifications.

The reader is advised to examine the various tables appended to this book so as to see if he can utilize them in his work and thus save himself some unnecessary labor in making computations.

CHAPTER II.

FIRST PRINCIPLES OF DESIGNING.

BOTH the student and the practitioner in bridge-designing will do well to recognize and bear constantly in mind certain first principles of design; and to enable them to do so, the author offers the following, which he considers will cover the essential fundamental principles that should govern the designing of all structural metal-work. Most of these will be repeated in the "General Specifications" given in Chapter XIV. under the heading "General Principles in Designing all Structures," for the reason that the said specifications would be incomplete without them.

The reason for this special chapter being devoted exclusively to these general principles is that the subject is of the utmost importance, and needs much more elaboration than could properly be given it in specifications. On this account the statement of each principle herein will be followed by remarks of an explanatory nature giving its *raison d'être* or application. It is to be noticed that the numbering does not agree with that of the "General Principles" in Chapter XIV.

The attention of the reader is called to the fact that this chapter is by far the most important one in the book, in that it contains in a concentrated form the most important conclusions drawn from the author's entire experience in his chosen specialty. The principles given have been established mainly by observation of the mistakes of others, and in a few cases, it must be confessed, by those of his own.

Few designers care to make public their errors for fear of the result being to their disadvantage; nevertheless far more is learned from the mistakes of construction than is learned in any other way.

The author would therefore suggest to the reader that he peruse this chapter carefully more than once before proceeding to the next.

PRINCIPLE I.

Simplicity is one of the highest attributes of good designing.

It is generally by means of a wide experience only that the young bridge engineer learns the truth of this assertion; but the older he grows and the more knowledge he acquires the more convinced does he become that simplicity, not only in design, but also in methods of execution of work, is one of the most important *desiderata*.

Other things being equal, that design which is the most simple, or contains the fewest parts, or involves the easiest connections, is the one which will be preferred by competent judges.

PRINCIPLE II.

"The easiest way's the best."

Although this principle was not enunciated originally in relation to structural metal-work, it nevertheless applies to it just as well, for the most successful engineer is he who in a given time can accomplish in a satisfactory manner the greatest amount of work.

This he can do only by the use of every labor-saving device of real value, by systematizing to the greatest practicable extent all that he does, and by making a thorough study of true economy of time and labor.

PRINCIPLE III.

The systemization of all that one does in connection with his professional work is one of the most important steps that can be taken towards the attainment of success.

Nor is this by any means all that can be said in favor of establishing a thorough system of doing work; because, in the first place, such a system enables one to accomplish a great deal in a very short time, and, in the second place, it is a subject of the greatest satisfaction and gratification to the man by whom it was evolved.

PRINCIPLE IV.

There is an inherent sense of fitness in the mind of a well-trained and well-balanced metal-work designer, which sense of fitness is of the greatest importance in all that he does.

It is this sense of fitness which enables him often, when inspecting the work of other designers, to see at a glance faults and flaws that would escape the observation of an untrained man. This faculty of rapid and correct judgment is one which can be developed, and one that should receive constant attention throughout an engineer's entire career. It is of special value in an office which employs a large number of draftsmen and computers, all of whose work has to be checked by the head of the office or by a reliable assistant. Nor is it only in connection with the work of others that this faculty is valuable, for it is often serviceable to an engineer on his own personal work, perhaps even without his being conscious thereof, saving him from making errors which pure theory might not enable him to detect, or which the authorities in his line have not yet recognized as errors. An example of this occurred some years ago in the author's practice which will serve to illustrate the point.

In proportioning reinforcing plates at pinholes, especially for hinged ends, the author has made a practice of instructing his draftsmen to extend these plates considerably beyond the length required by the theoretical number of rivets necessary for the connection, without his being able to give any valid or scientific reason for so doing. By some experiments made upon the ultimate strength of certain columns with hinged ends, the results of which were published in the *Transactions* of the Engineers' Society of Western Pennsylvania, Mr. Thomas H. Johnson has shown that such pin-plates, unless extended beyond the length required by the theoretical number of rivets, fail before the full strength of the compression-member is developed.

PRINCIPLE V.

There are no bridge specifications yet written, and there probably never will be any, which will enable an engineer to make a complete design for an important bridge without using his judgment to settle many points which the specifications do not properly cover; or as Mr. Theodore Cooper puts it: "The most perfect system of rules to insure success must be interpreted upon the broad grounds of professional intelligence and common sense."

At first thought one might conclude that this speaks badly for modern standard bridge specifications, and to a certain limited extent he would be right; for while it is quite true that no railway-bridge specifications yet published begin to cover the entire ground of ordinary bridge-designing at all adequately, or nearly as thoroughly as they might readily be made to do, nevertheless it is also true that the science of bridge-designing is such a profound and intricate one that it is absolutely impossible in any specification to cover the entire field and make rules to govern the scientific proportioning of all parts of all structures.

The author has done his best in Chapters XIV.–XIX. of this little treatise to render the last statement incorrect, but with what success time alone can prove.

PRINCIPLE VI.

In every detail of bridge-designing the principles of true economy must be applied by every one who desires to be a successful bridge engineer.

This subject is such an important one that to its consideration the whole of the next chapter will be devoted.

PRINCIPLE VII.

In bridge-designing rigidity is quite as important an element as is mere strength.

In fact each of these properties is dependent upon the other, because if a structure be amply proportioned in its main members for the assumed loads, but improperly sway-braced, the actual dynamic stresses will be greatly in excess of the live-load stresses provided for, and the metal will be overstrained in consequence; while, on the other hand, if rigidity be provided for by ample sway-bracing, but at the same time the main members of the structure be not adequately proportioned, the overstrained metal of the latter will cause vibration to be set up in spite of the sufficiency of sway-bracing. Both of these faults are to be found in existing structures. The effect of the first fault is usually the gradual wearing out of the structure by impact and rack, and that of the second, the sudden collapse of the bridge without previous warning.

Principle VIII.

The strength of a structure is measured by the strength of its weakest part.

This statement is as old as the hills, but is just as valid to-day as it ever was. The ignoring of its prime importance is constantly the source of waste of metal in structures, fundamentally weak in certain portions, by increasing the weights of other portions, and thus adding to the total load that the weak parts have to carry.

Principle IX.

In bridge-designing provision must always be made for the effect of impact, either by increasing the calculated total stresses by a varying percentage of the live-load stresses, or by decreasing the intensities of working stresses below those allowed for statically applied loads.

Different specifications accomplish this result differently. The former method is undoubtedly the more scientific and rational one, but the latter is the more common. The reason for this is that engineers, as a rule, dislike to specify various percentages to add to live loads for impact, when such percentages are established entirely by guesswork. An elaborate system of tests of actual intensities of working stresses for all main members of modern steel bridges under live loads, applied with varying velocities, is probably more urgently needed at the present time by the engineering profession than is any other series of experiments.

In the specifications of this treatise the effect of impact is provided for, how correctly only such experiments as those just referred to can demonstrate. As pointed out in Chapter I., the determination of the various amounts of impact was made solely by adopting a few fixed intensities of working stress and varying the percentages of impact so as to make the structures designed thereby agree as nearly as may be with the best general practice. If the impact formulæ adopted are ever proved to be incorrect, it will be a simple matter to correct them in a later edition.

Principle X.

In making the general layout of any structure, due attention should be given to the architectural effect, even if the result be to increase the cost somewhat.

There is no feature of bridge-designing which has been ignored in America to such an extent as has this ; and it is only of late years that even a few American engineers have paid any attention whatsoever to æsthetics in that branch of engineering. The subject is such an important one that to its consideration Chapter IV. will be specially devoted.

Principle XI.

For the sake of uniformity, and to conform to the unwritten laws of fitness, it is often necessary in bridge-designing to employ metal which is not really needed for either strength or rigidity.

The designer who recognizes this fact will usually produce structures of finer appearance than the designer who ignores it because of false notions of economy.

PRINCIPLE XII.

Before starting a design, one should obtain complete data for same.

If he fails to do so, he will generally have to make alteration after alteration as the work progresses; and, as one change usually entails several others, it will result that, by the time the work is finished, enough labor will have been expended thereon to complete two such designs, for which proper data were furnished at the outset.

PRINCIPLE XIII.

A skew-bridge is a structure the building of which should always be avoided when it is practicable.

It is generally possible to square the crossing either by swinging the centre line, or by lengthening the spans and squaring the piers or abutments. Sometimes, however, it is not practicable to do either, in which case the engineer must make the best of a bad business. The objections to a skew-bridge are these: First, it is fully twice as troublesome to design as a square structure; second, the liability to error in both shop and field is greatly increased by the skew; and, third, the resulting bridge is never so rigid, nor is it so satisfactory in a number of particulars, as a bridge without this objectionable feature.

PRINCIPLE XIV.

The best modern practice in bridge-engineering does not countenance the building of structures having more than a single system of cancellation, except in lateral systems where the resulting ambiguity of stress distribution is of minor importance.

Some engineer may question the correctness of this assertion; but if he will glance through the author's paper on "Some Disputed Points in Railway-Bridge Designing" published in the February and March, 1892, number of the *Transactions* of the American Society of Civil Engineers, he will see that, as a whole, the engineering profession indorses the statement. The only ordinary cases where multiple systems are employed nowadays are those of the lattice girder and the Whipple truss. The former is conceded by the leading bridge-designers to be unscientific, clumsy, often unsightly, and always uneconomical; and as for the latter, there is no longer any excuse for its use, because it has been ousted from the position it used to hold by the Petit truss, which excels it in every particular, including appearance, economy of material, and mathematical correctness.

PRINCIPLE XV.

The employment of a redundant member in a truss or girder is never allowable under any circumstances, unless it be in the mid-panel of a span having an odd number of panels, in which case, for the sake of appearance, two stiff diagonals can be used.

The reason for this is perfectly clear when one considers that it takes extra metal to build the said redundant member, and that its use upsets the calculations of stresses, rendering them in fact insolvable. A lengthy treatise was published a few years ago in India upon a method of finding stresses in redundant members, in which much good mental energy was wasted, for the entire book might have been written in these four words: "Never use such members." It is not often that an American engineer is found guilty of employing unnecessary pieces in his designs, but one cannot say the same of his European brethren.

PRINCIPLE XVI.

The use of a curved strut or tie in bridge-designing for the sake of appearance (or for any other reason) is an abomination that cannot for an instant be tolerated by a good designer.

It is hardly necessary to make such a forcible remark as this to American engineers, although in travelling about the United States one occasionally runs across a violation of the self-evident underlying principle involved in this statement; but the published records of some of the greatest bridges designed by English engineers show the use of pieces of trusses so curved that actually there is compression on one extreme fibre and tension on the other. Architectural effect is undoubtedly a very commendable feature in bridge-designing; but its adoption is no excuse for the violation of the fundamental principle that every compression or tension member of a truss or open-webbed girder should be absolutely straight from end to end.

It seems almost unnecessary to state that the appearance of curvature can be obtained by employing short panels and making each chord-length straight between panel points.

Principle XVII.

In all structural metal-work, excepting only the machinery for operating movable parts, no torsion on any member should be allowed if it can possibly be avoided; otherwise, the greatest care must be taken to provide ample strength and rigidity for every portion of the structure affected by such torsion.

It is not often that this question arises; nevertheless it is sometimes forced upon the consideration of the engineer. It came up lately in the author's practice in the case where an elevated-railroad exit-stairway, having at mid-height a landing and a 180-degree turn, had to be supported by a single column in order to comply with the demands of adjacent property owners.

Principle XVIII.

The gravity axes of all the main members of trusses and lateral systems coming together at any apex of a truss or girder should intersect in a point whenever such an arrangement is practicable; otherwise the greatest care must be employed to insure that all the induced stresses and bending moments caused by the eccentricity be properly provided for.

This is an important rule that is more often honored in the breach than in the observance; in fact, as far as the author knows, there are only a very few bridges in which the desired intersection in a single point of the axes of all members assembling at each apex is accomplished; and in most structures where eccentricity exists for want of such intersection, its prejudicial effects are not duly recognized and provided for.

Principle XIX.

Truss members and portions of truss members should always be arranged in pairs symmetrically about the plane of the truss, except in the case of single members, the axes of which lie in said plane of truss.

One occasionally sees a violation of this principle even in important bridges; but experience with structures in which it was ignored has been such as to show most clearly that this cannot be done with impunity, for the torsion resulting from eccentrically connected adjustable members is patent even to the uninitiated.

Principle XX.

In proportioning main members of bridges, symmetry of section about two principal planes at right angles to each other is a desideratum to be attained whenever practicable.

Of course in top-chord and inclined end-post sections, which should be designed with a cover-plate, symmetry about both principal planes is not attainable. The objectionable features caused by want of it, however, are provided against by the next axiom.

Principle XXI.

In both tension and compression members the centre line of applied stress must invariably coincide with the axial right line passing through the centres of gravity of all cross-sections of the member taken at right angles thereto.

Until very lately this important principle has been simply ignored, the effect being that the allowed intensities of working stresses are often exceeded by from fifty to one hundred per cent because of the eccentricity thus involved.

PRINCIPLE XXII.

The principle of symmetry in designing must be carried even into the riveting; and groups of rivets must be made to balance about central lines and central planes to as great an extent as is practicable.

The violation of this principle was exceedingly common not very long ago; and even to-day, when checking the shop drawings of some of the leading bridge-manufacturing companies. the author's assistants have to correct occasional departures therefrom.

PRINCIPLE XXIII.

In proportioning members of bridges to meet stresses and combinations of stresses it is important to consider duly the quality, frequency, and probability of the action of said stresses or combinations of stresses.

As a rule, standard specifications take care fairly well of this subject; nevertheless there will often occur in one's practice cases which they do not cover. The quality of stress should be considered in determining the sectional area of the member, because the greater the impact, other things remaining the same, the smaller should be the intensity of working stress. The frequency of application of stress should be considered, because, if a certain stress or combination of stresses be of frequent occurrence, a small intensity of working stress should be adopted, while for very infrequent occurrences the intensity can be taken considerably higher.

Finally, the probability of the application of a certain load or loads should be considered; because for inevitable loads or combinations of loads the metal should be strained fairly low, while for highly improbable loads or combinations of loads it is legitimate to strain much higher. Just here the author wishes to state most clearly and emphatically, that to indorse the points asserted under this heading one need not be a believer in the doctrines of Wöhler and Weyrauch, and in the theory of the fatigue of metals, because one's common sense will lead him to proportion sections of bridge members in accordance with the foregoing views.

In the specifications given in Chapters XIV. and XVI. the impact formulæ and the increased intensities for combinations of stresses involving those due to wind loads take care of this feature of design for all structures excepting high railroad trestles, in which latter the designer's professional judgment cannot well be eliminated.

PRINCIPLE XXIV.

In all main members having an excess of section above that called for by the greatest combination of stresses, the entire detailing should be proportioned to correspond with the utmost working capacity of the member, and not merely for the greatest total stress to which it may be subjected. In this connection, though, the reduced capacity of single angles connected by one leg only must not be forgotten.

It is almost needless to state that most engineers, especially those connected with contracting companies, will disagree with the author on the correctness of this statement; nevertheless he has yet to see the first case where adherence to the principle would involve improper, clumsy, or inappropriate construction. If it be right, for any reason, to use an extra amount of metal in the section of a member, why is it not also right to design that member throughout so that, if tested to destruction, it would fail as a whole and not in a detail? It seems to the author that the considerations which require extra section would demand either extra strength or extra rigidity, or both, in the details as well as in the section itself.

PRINCIPLE XXV.

In every bridge and trestle adequate provision must be made for the contraction and expansion of the metal.

Neglecting to comply with this principle has often been the cause of failure and disaster.

PRINCIPLE XXVI.

No matter how great its weight may be, every ordinary fixed span should be anchored effectively to its supports at each bearing on same.

At one end it should be anchored immovably, and at the other so as to provide for longitudinal expansion and contraction. Such anchorage prevents the dislodging of the structure by wind-pressure or by an accidental blow from a moving object.

PRINCIPLE XXVII.

The bridge-designer should never forget that it is essential throughout every design to provide adequate clearance for packing, and to leave ample room for assembling members in confined spaces.

There is no more fruitful source of profanity for bridge-erectors than the neglect of this principle; and as nearly every designer has to spend a year or two in learning to allow enough clearance, it follows that bridge-erectors should be given the benefit of "extenuating circumstances" when brought to judgment for their notorious addiction to the use of strong language.

PRINCIPLE XXVIII.

Although for various reasons engineers are agreed that field-riveting should be reduced to a minimum, such an opinion should not be allowed to militate against the employment of rigid lateral systems. All designs should be arranged so that the field-rivets can be driven readily.

One of the main reasons for the unsatisfactory condition of most of the elevated railroads of this country is that their designers endeavored in every possible way to avoid field-riveting, so as to keep down the cost of erection, and in so doing failed to develop the requisite amount of rigidity in the structures.

PRINCIPLE XXIX.

Rivets should not be used in direct tension.

In the days of iron rivets this was an important requirement, for the reason that the shanks were often so overstrained in cooling that the heads would fly off; but this does not occur with steel rivets. Nevertheless it is advisable to adhere to the rule, except for very unimportant members where there is a great excess in the number of rivets above the theoretical requirements

PRINCIPLE XXX.

For members of any importance two rivets do not make an adequate connection.

For such details as lattice bars, of course two rivets or even one rivet at each end will suffice; but where a direct calculable stress comes on the piece, and only two rivets at each end are used, it will be found that they will work loose, while, if three are used, they will not, unless they be overstrained by the calculated stress on the piece.

PRINCIPLE XXXI.

Designs must invariably be made so that all metal-work after erection shall be accessible to the paint-brush, except, of course, those surfaces which are in close contact either with each other or with the masonry.

This clause very properly cuts out the use of closed columns, which are a fruitful source of condemnation of old bridges.

PRINCIPLE XXXII.

In multiple-track structures, if any bracing-frames be used between panel points to connect the longitudinal girders of adjoining tracks, they must be designed without diagonals, in order to prevent the transference of any appreciable portion of the live load from one pair of girders to any other pair of same.

Such a transference would be doubly injurious; because it would throw on some of the girders more live load than they were proportioned to carry, and at the same time it would probably overstrain the diagonals and their connections, and would certainly tend to distort laterally the flange angles of the longitudinal girders.

PRINCIPLE XXXIII.

In bridges, trestles, and elevated railroads, the thrust from braked trains and the traction should be carried from the stringers or longitudinal girders to the posts or columns without producing any horizontal bending moment on the cross-girders.

This is a late requirement of the author's, that has been employed in his designs for a few years past. Its correctness was established in his before-mentioned paper on Elevated Railroads.

PRINCIPLE XXXIV.

In trestles and elevated railroads the columns should be carried up to the tops of the cross-girders or longitudinal girders and be effectively riveted thereto.

The correctness of this proposition also was established in the said paper on Elevated Railroads.

PRINCIPLE XXXV.

Every column that acts as a beam also should have solid webs at right angles to each other, as no reliance can be placed on lacing to carry a transverse load down the column.

The truth of this proposition is evident when one reflects that a single loose rivet or a single bent lacing-bar in the whole line of lacing will prevent the latter from carrying as a web a transverse load. Loose rivets and bent lacing-bars are, unfortunately, not uncommon in structural metal-work.

Principle XXXVI.

In trestles and elevated railroads every column should be anchored so firmly to its pedestal that failure by overturning or rupture could not occur in the neighborhood of the foot if the bent were tested to destruction.

As long ago as 1891 the author designed pedestals which involved truly fixed ends for column feet; but it is only within the last three years that such a detail has begun to come into general use. The ordinary connection of columns to pedestals by an anchor-bolt at each of the four corners of the bed-plate is extremely weak and ineffective.

Principle XXXVII.

All pedestals for trestles, viaducts, and elevated railroads should be raised to such an elevation as to prevent the accumulation of dirt and moisture about the column feet, and all boxed spaces in the latter should be filled with extra-rich Portland-cement concrete.

The neglect of these precautions causes the rapid deterioration of the metal at bases of columns, and thus shortens the life of the structure.

Principle XXXVIII.

In designing short members of open-webbed, riveted work, it is better to increase the sectional area of the piece from ten to twenty-five per cent than to try to develop the theoretical strength by using supplementary angles at the ends to connect to the plates.

This principle is based upon the results of some late tests of the author's on the strength of single angles and pairs of angles connected by one leg only, by which he found that $6'' \times 3\frac{1}{2}''$ angles thus connected developed ninety per cent of the ultimate strength of a flat bar of equal net section, and that $3'' \times 3''$ angles developed seventy-five per cent of same.

Principle XXXIX.

Star-struts formed of two angles with occasional short pieces of angle or plate for staying same do not make satisfactory members. Better results are obtained by placing the angles in the form of a T.

The truth of this statement was established by another series of experiments of the author's made at the same time as were the last-mentioned tests. The specimen columns did not develop on the average more than seventy-five per cent of the resistance they should have developed according to the usual straight-line formula for metal of the same tensile strength.

Principle XL.

In making estimates of weights of metal the computer should always be liberal in allowing for the weight of details.

It is the author's experience that, in nearly every case, the weight of the finished structure exceeds slightly the estimated weight, and mainly on account of the use of more metal for details than was figured upon. Of course, if one sets out deliberately to "skin" a bridge so as to save all the metal he can, the actual weights of details may be made to underrun the estimate; but such a practice is most reprehensible.

Principle XLI.

In general details must always be proportioned to resist every direct and indirect stress that may ever come upon them under any possible condition, without subjecting any portion of their material to a stress greater than the legitimate corresponding working stress.

This principle, which has been given before in several of the author's previous works on bridges, involves the whole theory of bridge detailing.

PRINCIPLE XLII.

There is but one correct method of checking thoroughly the entire detailing of a finished design for a structure, viz.: "Follow each stress given on the stress-diagram from its point of application on one main member until it is transferred completely to either other main members or to the substructure, and see that each plate, pin, rivet, or other detail by which it travels has sufficient strength in every particular to resist properly the stress that it thus carries; check also the sizes of such parts as stay-plates and lacing, which are not affected by the stresses given on the diagram, and see that said sizes are in conformity with the best modern practice."

But to do all this as it should be done necessitates the computer's being, in the highest sense of the term, an "expert on structural metal-work."

CHAPTER III.

TRUE ECONOMY IN DESIGN.

TREATISE after treatise has been written upon the subject of economy in superstructure design, but unfortunately the result is simply a waste of good mental energy; for the writers thereof invariably attack the problem by means of complicated mathematical investigations, not recognizing the fact that the questions they endeavor to solve are altogether too intricate to be undertaken by mathematics. The object of each investigation appears to have been to establish an equation for the economic depth of truss, or that depth which corresponds to the minimum amount of metal required for said truss; and, to start the investigation, it seems to have been customary to make certain assumptions which are not even approximately correct. For instance, the principal assumption of several treatises in French and English is that the sectional area and the weight of each member of a truss are directly proportional to its greatest stress; or, in other words, that in proportioning all members of trusses a constant intensity of working stress is to be used, while in reality for modern steel bridges the intensities vary from, say, 6000 pounds up to 15,000 pounds, or, when impact is provided for, up to 18,000 pounds, and when both impact and wind stresses are included, up to nearly 24,000 pounds. Again, no distinction is made between tension and compression members, and no account is taken of the greatly varying amounts of their percentages of weights of details.

There is, however, one mathematical investigation concerning economic truss depths which, in the author's opinion, is approximately correct, and which is based on assumptions that are very nearly true · but it holds good only for parallel chords. It is this:

Let A = weight of the chords,
B = weight of the web,
C = weight of the truss,
and D = depth of truss.

Then
$$C = A + B.$$

But the weight of the chords varies inversely as the depth, or $A = \frac{a}{D}$, and the weight of the web varies directly as the depth, or $B = bD$, where a and b are constants; and therefore

$$C = \frac{a}{D} + bD.$$

If C is to be made a minimum, we shall have, by differentiation,

$$\frac{dC}{dD} = -\frac{a}{D^2} + b = 0,$$

or
$$-\frac{A}{D} + \frac{B}{D} = 0, \quad \text{or} \quad A = B.$$

As the second differential coefficient, after substitution according to the usual method for maxima and minima, comes out positive, the result obtained corresponds to a minimum.

From this it is evident that, for trusses with parallel chords, the greatest economy of material will prevail when the weight of the chords is equal to the weight of the web. The author has verified this conclusion by checking the weights of chords and webs in a number of finished designs, finding it to be absolutely reliable. However, it is not of much practical value, because the economic depths of trusses with parallel chords are pretty well known; and, again, when spans are in excess of 175 or 200 feet, the chords of through-bridges are seldom made parallel. Moreover, the best depth to use is not often the one which gives the least weight of metal in the trusses.

The author finds by experience that, for trusses with polygonal top chords, the economic depths, as far as weight of metal is concerned, are generally much greater than certain important conditions will permit to be used. For instance, especially in single-track bridges, after a certain truss depth is exceeded, the overturning effect of the wind-pressure is so great as to reduce the dead-load tension on the windward bottom chord to such an extent that the compression from the wind load carried by the lower lateral system causes reversion of stress, and such reversion eye-bars are not adapted to withstand. A very deep truss requires an expensive traveller, and to decrease the theoretically economic depth increases the weight but slightly; hence it is really economical to reduce the depth of both truss and traveller.

Again, the total cost of a structure does not vary directly as the total weight of metal, for the reason that an increase in the sectional area of a piece adds nothing to the cost of its manufacture, and but little to the cost of erection; so it is only for raw material and freight that the expense is really increased. Hence it is generally best to use truss depths considerably less than those which would require the minimum amount of metal. For parallel chords, the theoretically economic truss depths vary from one fifth of the span for spans of 100 feet to about one sixth of the span for spans of 200 feet; but for modern railway through-bridges the least allowable truss depth is about 28 feet, unless suspended floor-beams be used, a detail which very properly has gone out of fashion.

In two five-hundred-foot spans of a combined railway and highway bridge the author employed a truss depth of seventy-two feet; but this was determined by the reversal of stress in bottom chords through wind-pressure. A greater depth, if permissible, would have caused a saving in total weight of metal.

In a design of the author's for a five-hundred-and-sixty-foot span a truss depth of ninety feet was adopted, but in this case the live load was very great, varying from ten thousand pounds per lineal foot for short spans to eight thousand pounds per lineal foot for long spans; and the bridge is twenty per cent wider than in the case of the two five-hundred-foot spans just mentioned.

The greater the live load and the wider the bridge, the greater can the truss depth be made advantageously.

The little mathematical investigation given in this chapter can be applied with advantage to plate-girder bridges and to the floor systems of truss-bridges. If for ordinary cases, in designing plate girders, one will adopt such a depth as will make the total weight of the web with its splice-plates and stiffening angles about equal to the weight of the flanges, he will obtain an economically designed girder, and a deep and stiff one. For long spans, however, this arrangement would make the girders so deep as to become clumsy and expensive to handle; consequently when a span exceeds, say, forty feet, the amount of metal in the flanges should be a little greater than that in the web; and the more the span exceeds forty feet the greater should be the relative amount of metal in the flanges.

Concerning economic panel lengths, it is safe to make the following statement: "Within the limit set by good judgment and one's inherent sense of fitness, the longer the panel the greater the economy of material in the superstructure." Of course, when one goes to such an extent as to use a thirty-foot panel in an ordinary single-track bridge he exceeds the limits referred to, because the lateral diagonals become too long, and their inclination to the chords becomes too flat for rigidity. Again, an extremely long panel would often cause the truss diagonals to have an unsightly appearance, because of their small inclination to the horizontal.

There is another mathematical investigation which is of practical value. It relates to the economic lengths of spans, and was first demonstrated, in print, by the author some six years ago in *Indian Engineering*, although the principle was announced three years before then in the first edition of his *General Specifications for Highway Bridges of Iron and Steel.* Strange to say, many engineers failed to see that there is any difference between this principle and an old practice of forty years' standing. The principle is that "for any cross ing the greatest economy will be attained when the cost per lineal foot of the substructure is equal to the cost per lineal foot of the trusses and lateral systems." The old practice was to make for economy the cost of a pier equal to the cost of the span that it supports, or, more properly, equal to one half of the cost of the two spans that it helps to support.

Is not the difference between these two methods perfectly plain? In one the cost of the pier is made equal to the cost of the trusses and laterals, and in the other it is made equal to the cost of the trusses, laterals, and the floor system. When one considers that the cost of the floor system is sometimes almost as great as one half of the total cost of the superstructure, he will recognize how faulty the old method was.

The following is the demonstration of the principle, simplified to the greatest practicable extent. Let us assume a crossing of indefinite length, for which the depth of bed-rock is constant, and let

S = cost per lineal foot of the substructure,
T = cost per lineal foot of the trusses and laterals
F = cost per lineal foot of the floor system,
B = cost per lineal foot of the entire bridge,
and L = length of span;

then
$$B = S + T + F.$$

Now if we assume that slight changes in length of span will not affect materially the sizes of the piers, the cost per foot of the substructure will vary inversely as the span length,

or
$$S = \frac{s}{L}.$$

Again, the cost per foot of the trusses and laterals, for slight changes in length of span, may be assumed to vary nearly directly as the span length; hence we may write the equation

$$T = tL.$$

The cost per foot of the floor system is practically independent of the span length, being a function of the panel length, which does not change materially with the span. We now have the equation

$$B = \frac{s}{L} + tL + F,$$

in which B is to be made a minimum.

Differentiating, we have (as F is a constant)

$$\frac{dB}{dL} = -\frac{S}{L} + \frac{T}{L} = 0, \quad \text{or} \quad S = T.$$

A further differentiation shows that the result corresponds to a minimum.

In reality the truss weight per foot increases more rapidly than the span length. If r is the ratio of span lengths, the truss weights, for small changes in span lengths, will vary almost exactly according to the ratio $r^1 = \frac{1}{2}(r + r^2)$. On the other hand, the weight per foot for the lateral system does not increase as rapidly as the span, unless the perpendicular distance between central planes of trusses also increases. Unfortunately, though, the gain in truss weight over that given by the assumed theory of variation is generally greater than the corresponding loss for the weight of lateral system, consequently the combined weights per foot of trusses and laterals generally increase a trifle faster than the span length. This is partially offset by the fact that the pound price of metal erected and painted will reduce a trifle as the weight per foot increases.

Again, there is sometimes a small error in the assumption that the cost of the piers varies inversely as the span length, because the size of each pier may have to be increased a little to accommodate the heavier spans. If the perpendicular distance between central planes of trusses is increased because of the greater span length, the cost of each pier will be increased because of its greater length; but this will occur only occasionally.

Ignoring the latter contingency, the two errors indicated, notwithstanding the fact that their effects are additive, are so small as not to affect materially the correctness of the results of this investigation concerning economic span lengths.

This demonstration proves that, in any layout of spans, with the conditions assumed, the greatest economy will be attained when the cost of the substructure per lineal foot of bridge is equal to the cost per lineal foot of the trusses and lateral systems. Of course no such condition as a bridge of indefinite extent ever exists, nor is the bed-rock often level over the whole crossing; nevertheless the principle can be applied to each pier and the spans that it helps to support by making the cost of each pier equal to one half of the total cost of the trusses and laterals of both spans. Since working out this demonstration some ten years ago, the author has made a practice of checking the correctness of the principle thereby established, by comparing the cost of substructure and superstructure in a number of bridges which he has designed and built, with the result that he finds it to be exact.

The principle will apply also to trestles and elevated roads, for in the latter, if we make the cost of the stringers or longitudinal girders of one span equal to the cost of the bent at one end of same, including its pedestals, we shall obtain the most economic layout. In an ordinary railroad trestle, consisting of alternate spans and towers, it will be necessary for greatest economy to have the cost of all the girders in two spans (one span being over the tower) plus the cost of the longitudinal bracing of one tower, equal to the cost of the two bents of said tower, including their pedestals.

On page 235 of the first edition of Prof. J. B. Johnson's "Theory and Practice of Modern Framed Structures," Mr. Bryan uses this method of the author's in a slightly different form for determining the most economic number of spans to adopt at any crossing, establishing the equation,

$$y = A + B + (x - 1)C + l\frac{al}{bx}p,$$

in which y (the total cost of bridge) is a minimum when

$$\frac{l}{x} = \sqrt{\frac{b}{ap} C},$$

where A = cost of two end abutments *in dollars;*
B = cost of the floor and that part of the metal weight which remains constant, *in dollars;*
C = cost of one pier in dollars, assumed as constant;
l = length of bridge in feet;
x = number of spans;
p = price of metal per pound, *in dollars;*
y = total cost of bridge *in dollars;*
and a = weight per foot of a span b feet in length.

Thus far all right; but then he makes an assumption which will not be correct except for one live load, for one set of specifications, and for single-track railway bridges, viz., that for pin-connected spans

$$\frac{b}{a} = \frac{1}{5}.$$

On account of this assumption his subsequent table of economic span lengths is not in any sense general, but is true only for single-track bridges designed for one standard live load and according to one standard set of specifications; while his equations hold good for bridges of any kind and loading, including highway as well as railway structures.

As a check on the correctness of Mr. Bryan's assumption that $\frac{b}{a} = \frac{1}{5}$ for single-track bridges, the author has looked up some of his designs and has found the following:

For a 375-ft. through-span, Class X, $\frac{b}{a} = \frac{1}{5.7}$; for a 362-ft. double-track through-span, Class Z, $\frac{b}{a} = \frac{1}{9.7}$; and for a similar 490-ft. span, $\frac{b}{a} = \frac{1}{9.0}$. For a 280-ft. double-track deck-span, Class Y, $\frac{b}{a} = \frac{1}{9.1}$, and for a similar 200-ft. deck-span $\frac{b}{a} = \frac{1}{9.5}$. For a single-track 200-ft. through-span, designed by a contracting bridge company and checked by the author, $\frac{b}{a} = \frac{1}{4.7}$. The detailing thereof, however, was ultra-economical. It is but fair to state that the 375-ft. span is about two feet wider than the ordinary single-track bridges of such span lengths, which causes the denominator of the fraction to increase somewhat. It is evident, though, that the assumption of any fixed value for $\frac{b}{a}$ is unwarranted, because the weights per foot of trusses and laterals for spans of Classes Z and U of the Compromise Standard System will vary by from, say, 33 to 40 per cent, according to the span length; consequently the values of $\frac{b}{a}$ would vary likewise.

In cases of structures for crossings where there is danger from washout, it may be truly economical to use metal unsparingly in the design, in order to ensure the metal-work going together readily and with the least possible delay; and in extreme cases it would be eminently economical to adopt a cantilever design, and thus reduce the risk of washout to a minimum by the expenditure of a considerable amount of extra metal for the superstructure.

There is another economic feature of design, which, unfortunately, has been overlooked continually, viz., that the most economic structure is the one for which the first cost, plus the capitalized cost of annual deterioration and repairs, is a minimum. A proper consideration of this economic feature would cause the use of better details, larger sections of main members, more efficient and rigid sway-bracing, and a greater minimum thickness of metal.

CHAPTER IV.

ÆSTHETICS IN DESIGN.

THAT the metal bridges built in the United States during the last two or three decades are, with rare exceptions, anything but models of excellence in respect to the principles of æsthetics, no engineer is likely to deny. For this the principal reasons are as follows:

First. Very few technical schools in this country instruct their engineering students at all in architecture; and not one of them gives to that important branch of constructive engineering the attention it merits.

Second. As most American enterprises are consummated with a small amount of money compared with what might be spent advantageously in their materialization and completion, there are seldom any funds to employ in decorating the work.

Third. American engineers, as a rule, appear to regard with more or less contempt all efforts to ingraft architectural ideas upon engineering construction. While the engineering profession is only too ready to criticise architectural construction because of its numerous violations of the principles of engineering practice, it does not appear to see that the converse of the proposition holds good, viz., that the architectural profession has good reason to criticise severely engineering construction in general because of its numerous and glaring violations of the principles of architecture. Moreover, in no branch of engineering are such violations so common and so pronounced as in that of bridge building.

Fourth. But the chief factor, the one which has had more bad influence than all the others combined, is the custom of letting bridges upon competitive designs and awarding the contract to the lowest bidder.

For many years prominent architects have very justly inveighed against the inherent ugliness of American bridges. In order, therefore, to see what such violations of æsthetics in bridge-designing really are, and to what extent they can be avoided, the author has asked his friend, Henry Van Brunt, Esq., of the architectural firm of Van Brunt & Howe, who is acknowledged by the leading members of his profession to be one of the foremost living masters of the science of architecture, to write for publication in this treatise a letter formulating the charges of himself and his professional brethren against the bridge-builders of this country in respect to their alleged offences against the æsthetics of construction. In response to this request Mr. Van Brunt has written the following letter:

MY DEAR MR. WADDELL:

After looking over a portion of your instructive treatise on bridges, I find it quite impossible to comply with your request to furnish you with practical suggestions from an architectural point of view as to grace and beauty of design in such structures. As these qualities must be developed from the structure itself, as they must be evolved from its inherent economical and practical conditions, and as they cannot be successfully applied to it as an afterthought, it would be unbecoming for any layman to attempt to show by what process this evolution is to be accomplished. The problem is not an easy one; it is not to be solved by theory, or by any accident of invention or ingenuity. At present, at least, it can only be treated on general lines. Indeed there is no one living, I fear, who can suggest a specific and easily applied remedy for that disease of engineering which is expressed in the curious fact that the most perfect results of science, at least in the art of steel-bridge building as now understood and inculcated, do not recognize any theory of beauty in line or mass.

It is the business of the architect to express structure and purpose with beauty. It is the business of the engineer, as I understand it, to make structures strong, durable, rigid, and economical; to apply pure science, excluding, as a matter of principle, any device of art which, for the sake of mere ornamentation, may add to his fabric a pound of unnecessary weight or a dollar of unnecessary cost.

It cannot be denied that to whatever extent the exercise of this principle may have affected the practice of engineers, they have succeeded, especially as regards bridge-building, in developing a structure which is in every essential respect orderly, consistent, and progressive from a practical point of view. From year to year this development towards mechanical perfection has been plainly visible. The structure of ten years ago has been reasonably and properly superseded by another and better structure, indicating a process of growth without a shadow of caprice; in this process discovery and invention have had their proper influence, uninterrupted by any conservative prejudice or by any theory of design which does not rest directly on practical considerations. But, as I have already observed, this admirable and prolific progress has not carried with it a corresponding progress in grace and beauty of design. In fact, these qualities seem to appear in an inverse proportion to the development of the structural scheme towards the practical idea of strength, stability, and economy. Consequently the stronger, the more rigid, the more economical the structure, the more uncompromising and the more hopeless it seems to be in respect to beauty. The modern steel-girder or cantilever bridge, while, according to our present knowledge, it is perfectly adapted to its uses and functions, is in nearly every case an offence to the landscape in which it occurs. Its lines, since they have ceased to be structural curves, have become hard and ascetic mathematical expressions, and have not been brought into any sympathy whatever with the natural lines of the stream which it crosses, of the opposite banks which it connects, of the meadows, forests, and mountains among which it is placed. All sylvan effects of harmony are shocked by its discordant intrusion. The vast aqueducts of the Romans, the arched bridges of stone, the catenary curves of the modern suspension bridges with their high towers, and some forms of bridges constructed with bowstring girders, are more or less affiliated with the natural conditions, so that they give no shock, save frequently of pleasure at their expression of grace and fitness. But we are assured that these structural forms are obsolete or are becoming obsolete, and that the straight bridge-truss spanning from pier to pier, the cantilever overhanging the perilous abyss, the pivoted draw-span, all constructed with cold geometrical precision, with hard unfeeling lines of tension and compression, have taken their place, to the great advantage of the railroads and the greater security of the public. It is in vain that the conscientious engineer occasionally attempts to compromise with grace by ornamenting his intersections by rosettes or buttons of cast iron, or by rearing a sort of arch or portal of triumph at the entrance to his bridge with a lavish display of metal shell-work, scrolls of forged iron, and tables cast and gilded with names and dates. But the compromise comes too late; the main essential lines cannot be condoned by afterthoughts of this sort; and as far as the eye can see, these lines, though they may satisfy the reason, generally affront the sense of beauty.

Now it seems to me important to note that the methods of nature always culminate in infinite expressions of beauty, and that beauty is an essential part of the principles of natural growth. The Great Creator never makes anything, animate or inanimate, ugly in making it strong or swift or durable, or in fitting it to the economy of nature. Grace is a part of the system of creation. Is it reserved for man in his secondary creation to make things unlovely in proportion to their complete and perfect adaptation to the satisfaction of his practical needs? Is this difference significant of some quality which is wanting in our science?

But, it may be said, if a steel-trussed bridge, economically and wisely constructed according to our present light, offends our ideals of grace and beauty, the fault perhaps is not in the structure, but in the rigidity and immobility of the ideals which have been established by conditions long since outgrown in the progress of science. The attempts of the English bridge-builders in iron in the early part of the century to meet these old ideas resulted in constructions which, though they may satisfy the eye of the artist, and combine more or less gracefully with the landscape, are uneconomical and unscientific. The principles of structure involved are incorrect, and unnecessary expense was incurred in forcing into the design features conventionally acceptable, but which had nothing to do with the structure, and which in fact were a hindrance to it, concealing rather than illustrating it.

The architect will not find it difficult to agree with his brother the engineer, that a mask of ornamental cast iron, covering the essential features of the structure in order to force upon it an effect of grace, is illogical in the extreme. Indeed, a great modern master of architecture has laid down the axiom: "A form which admits of no explanation, or which is mere caprice, cannot be beautiful; and in architecture, certainly, every form which is not inspired by the structure ought therefore to be rejected." The conscientious modern architect aims to shape his design according to this reasonable limitation, and he has been thereby enabled to produce occasional effects of beauty without imposing on his composition a single idea which is not suggested either by the structure or by the use of the building. Even a factory, a gasometer, a railway shed, an elevator, need not challenge the architect in vain to produce effects of fitness not entirely inconsistent with the requirements of art. Indeed, the engineer himself, with axioms or maxims of art, has, in the evolution of the roof-truss, the locomotive, and many industrial machines, succeeded in satisfying ideals of beauty in the very process of making them powerful, compact, and economical of material and space. The modern steel-armored war-ship has already, in this early stage of its rapid development, substituted for the ideas of maritime beauty, speed, and strength which prevailed in the time of Nelson and the other great historical admirals, and which were celebrated in the songs of Dibdin and Campbell, an entirely different ideal, hardly less imposing, though as yet without poetic recognition. But the evolution of the steel-trussed bridge has as yet satisfied neither old ideals of beauty, nor has it made new ideals. Its essential lines are drawn in apparent disregard or contempt for grace of outline or elegance of detail. The difficulty seems to be inherent in the present approved structural system of designing horizontal, straight, open-trussed girders or cantilevers, resting on rigid vertical piers of masonry or iron, without regard to any other considerations excepting those of statics. The eye requires to be satisfied as well as the trained intelligence, and demands not only grace of proportion, but a certain decorative emphasis expressive of especial functions. The primitive post and lintel structure of stone was as hopeless, apparently, as its modern derivative, the steel-trussed bridge, until the Greeks, with unerring instinct of art, converted it by perfectly rational processes into that ideal expression of beauty which is known as the Doric order. This Doric order is a structure which depends less upon subsidiary decoration than upon proportion for its unparalleled success as a work of art. The Parthenon would still be lovely without the sculptures of its friezes, metopes, and pediments. Its columns, reduced to dimensions which encumber them with no useless brute mass of material, were so treated with entasis, capital, and fluting as to express exactly members in vertical compression; its lintels were so subdivided as to draw attention to, and to illustrate, all their functions in the structural scheme. They contained no features of caprice or fancy. Now the essential qualities of the steel-girder bridge differ from those of the post and lintel of the Greeks because, in the former, the structure of the lintels permits of a wider spacing of the posts, and the posts have assumed the dual function of piers for vertical support and of buttresses to withstand the horizontal pressures of the stream in which they are built; the lintels, in their turn, have lost their quality as compact, solid, homogeneous masses, have been resolved into distinct elements, and have become a complicated and highly artificial openwork contrivance of light steel members, which in their dimensions and articulations have been so combined in tension and compression as to produce a structure capable of sustaining without change of form not only its own weight between bearing points far apart, but that of moving trains, and of bearing without detriment vibrations and wind-pressures, and the expansion and contraction of its material by changes of temperature.

These compound lintels or trusses are in themselves triumphs of mind over matter. At this moment they express a stage of evolution which has been in process for a century, and which doubtless will continue to develop in directions impossible to anticipate. They are structures not dedicated to the immortal gods, like the post and lintel in the Greek temples, the decorative character of which was largely inspired by religious emotions, but devised to meet secular and practical conditions of an exceedingly unpoetic and unimaginative character. The mind of the architect appreciates the fine economy of these sensitive and complicated organisms, but it also recognizes that they are still in active process of development; that they are on trial, and will not reach final results *until they shall have assumed those conditions of grace and beauty which are essential to completion*. It is evident enough that all the features of perfection in animals have been very gradually evolved, by survival of the fittest and by adaptation to use, from the awkward and monstrous shapes of the antediluvian period; that geological erosion and drift have clothed the naked rocks with beauty; and that the whole vegetable creation has been improved by art. Nature herself is not contented with inelastic dogmas. In like manner, the locomotive, the steam-engine, the modern war-ship, have all become objects of awful beauty, not because of the imposition of unnecessary features, but because of the natural and reasonable growth of their essential structure.

If, therefore, the ugly character of the present steel-trussed bridge is in itself a proof of the immaturity of the science which has produced it, the remedy, of course, must reside in the perfecting of the science, and this process of perfecting will be quickened, if beauty is recognized in engineering as it is in architecture, as an aim and not as an accident of growth. The architect guides and hastens this progress towards the perfect type by fundamentally composing his structure with a view to an agreeable proportion of its parts; in detail he studies to emphasize the special and important points of his structure by a decorative treatment which shall indicate conventionally the character of the work accomplished at these points. It is true, perhaps, that the structural forms of materials with which the engineers have to work, especially in bridge-building, are hardly so elastic and manageable as those at the command of the architect even in his simplest and most severely practical problems; but it is none the less true that the training of the engineer leads him too often to an absolute disregard, if not contempt, for those refinements of proportion and outline, and for all those delicate adaptations and adjustments of detail, which, though perhaps separately slight, and apparently of small importance, in combination tend to give distinction and a character of fitness and grace to works otherwise, from the point of view of art, rudely immature, basely mechanical, unnecessarily and insolently ugly.

Mr. Henry James says that the French talk of those who see *en beau* and those who see *en laid*. The performance of the modern steel-bridge designers would certainly seem to place them in the latter category. It is no less certain that this result comes not from temperament, which is natural, but from training, which is artificial. The severe and absolute conditions in which the bridge-builders work do not prevent them either from great differences in manner and method of design, or from frequent and unnecessary extravagances of expenditure; but these extravagances are rarely, if ever, lavished in the services of beauty; because the cold and rarefied atmosphere of science and mechanical utility, in which they are accustomed to labor, has gradually frozen out the finer natural instinct which works for art and elegance in design. Beauty of proportion has often been proved by mathematics; but mathematics, when it has been allowed to be the only element in the development of a problem of construction, has never accomplished beautiful results. Such results do not come by accident in any work of design, but by the liberal and generous observance of natural laws. The education, therefore, which from the beginning does not give some recognition to grace, proportion, elegance, as essential parts of construction, must be misleading and one-sided, and cannot lead to perfection. The recognition of these qualities, I am entirely persuaded, does not necessarily imply any sacrifice of practical accuracy in design or of mechanical precision in workmanship, nor need it affect materially that fine economy which is essential to perfection.

Very sincerely yours,

HENRY VAN BRUNT.

This letter of Mr. Van Brunt's, in the author's opinion, gives a very just and unprejudiced statement of the status of affairs in relation to the development of bridge-building from the æsthetic point of view; and, in calling the attention of bridge-designers to their lamentable indifference towards beauty in construction, it ought to be the means of inaugurating a much-needed reform in bridge-designing.

In thus candidly acknowledging the correctness of these allegations of the architectural profession against the work of American bridge-designers the author wishes it to be understood that he considers a large portion of his own past work as properly subject to censure; but that for several years, more especially since he severed all connection with the contracting branch of bridge-building, he has been endeavoring to reform in this particular, and with a certain amount of success, interspersed perhaps with more or less of failure.

The principal hindrance to the progress of æsthetic reform in bridge-building is liable to emanate from the bridge-manufacturing companies, who have been so accustomed to submitting competitive designs, and who have made in the past so much money thereby, that they will naturally consider any fundamental innovation of this kind as detrimental to their interests. Nevertheless, when some concerted action on the part of bridge specialists is inaugurated with the object of making bridge structures more sightly, it is probable that the manufacturing companies will be far-sighted enough to recognize that their true interests will not be subserved by offering any serious opposition to the proposed reform. Some obstruction is likely to come from managers of railroads, who have for years been used to buying their bridges as cheaply as possible without any regard to appearance, and too often with very little in respect to constructive excellence. It will devolve upon the chief engineers and the bridge engineers of railroads to influence the managements of their lines so as to incline them towards a more favorable consideration for appearance when deciding upon the designing and purchasing of their bridges.

But the moulders of public opinion in respect to the necessity for a due consideration of architectural effect in bridge-building must of necessity be the independent bridge engineers of the country, who are not so much influenced by monetary motives as are engineers connected with railways and bridge companies, although it must be confessed that some of the most prominent bridge specialists are the greatest offenders against the principles of æsthetics.

There is a general impression among engineers that to ingraft architectural effects upon bridge construction will always involve the necessity for an increased expenditure of money; but this notion is incorrect, because there are many large and important bridges in the United States which could have been beautified, and at the same time cheapened, without in the slightest degree impairing their strength, rigidity, or efficiency, by simply modifying their harsh and uncompromising lines. It requires the expenditure of more thought than money to obtain an artistically designed bridge; for a little money will go a long way in producing a decorative effect upon such a structure.

Distinction must be made between appropriate and inappropriate, necessary and unnecessary, and expensive and inexpensive decoration. For instance, while it is always proper to adapt the lines of a structure to the production of the most graceful effect, provided that in so doing no sacrifice of constructive excellence be thereby involved or extra expense incurred, it would often be injudicious to expend money on pure decoration. The builder probably cannot spare the money, and the location of the structure may be such that any extra expense for ornamentation would be absolutely wasted. If a bridge is to be located where it will be seen constantly by many people, it is well to spend extra money to make it sightly, beautiful, and in keeping with its surroundings; but when it is to be placed in a dense forest or on a sandy desert where it would seldom be seen, it would be folly to spend any more on its construction than is called for by the engineering requirements of the conditions, due allowance being made, of course, for a possible peopling of the forest or desert in the not very distant future.

The style of ornamentation for a bridge should always be in keeping with its general character; thus, in case of a light highway bridge, ornamental portals with filigree metal-work are appropriate, while in large, massive railway bridges the ornamentation should be of a coarser and bolder character, commensurate with the size and use of the structure.

The author is a firm believer in the principle that true economy, engineering excellence of construction, and the best architectural effect will almost invariably be found to accompany each other, and be inseparable in the designing of any bridge. Moreover, any bridge built with due consideration for, first, efficiency, second, appearance, and, third, economy, will be satisfactory and gratifying to not only the trained expert, but also to the general engineer and railroad man, and even to the public; because when an observer notes that in such a structure all the engineering requirements are properly provided for, that there is no evident waste of material, and that all due advantage has been taken of the conditions to render the bridge sightly and in harmony with its surroundings, his eye will of necessity be pleased, and his inherent sense of fitness will cause him to regard the structure with a feeling of pleasure.

In suggesting that "if a steel trussed bridge, economically and wisely constructed according to our present light, offends our ideals of grace and beauty, the fault perhaps is not in the structure, but in the rigidity and immobility of the ideals which have been established by conditions long since outgrown in the progress of science," Mr. Van Brunt has probably indicated the lines of convergence of engineering practice and architectural ideals; for while, as before stated, much can be done with most bridge designs to improve them without increasing their cost or affecting their efficiency, on the other hand it is often impossible for an engineer to modify a bridge design so as to meet fully the critical objections of a good architect without introducing features both faulty and expensive. It is highly probable that if the engineer will modify his designs as much as is legitimate to meet the æsthetic requirements of the architect, the latter will gradually modify the rigidity of his ideals, and will see lines of grace, beauty, and fitness in the polygonal outlines of trussed bridges. Mr. Van Brunt himself has already shown this to be true by giving his unqualified approval to the architectural effect of the truss outlines in the draw-span of the author's bridge over the Missouri River at Omaha, although these outlines were determined primarily for utility and secondarily for appearance, and notwithstanding the fact that there is no attempt at even approximate curvature of chords in the entire span.

To recognize and acknowledge the deficiencies of modern bridge designs from the artistic point of view is one thing, but to show how they are to be remedied is another; because, while it is easy to say that a certain structure does not come up to one's ideal of grace and beauty, it is very difficult to show exactly where the defects are, and what should or can be done to remove them.

Notwithstanding this, the author will now endeavor to establish a few fundamental rules which, if followed, ought to correct the most glaring sources of ugliness in bridge designs; then, by entering more into detail, he will try to show how the structures may be decorated appropriately and inexpensively.

The architectural treatment of bridge-designing may be divided into these four parts:

1st. The layout of spans, piers, and approaches.

2d. The outlining of each span.

3d. The decoration of each span.

4th. The ornamentation of the entire structure by the adoption of elaborately artistic approaches.

In respect to the layout of spans, piers, and approaches for any bridge, there is one governing principle which should always be complied with, viz., that the entire structure, whenever possible, should be made perfectly symmetrical about a middle plane.

There is no feature of a bridge so pleasing to the eyes of all observers, cultivated and ignorant alike, as perfect symmetry in the layout of spans; consequently it should be attained whenever practicable, even if some extra expense be involved thereby.

Unfortunately the conditions are not always favorable to perfect symmetry of design, for the bed-rock will often dip rapidly, and thus necessitate the use of spans of different lengths, and the channel of the river often refuses to keep at midstream, persisting in hugging one shore. In such cases it becomes necessary to do the best one can with the unfavorable conditions, and to make the structure sightly, if not symmetrical. If there be a draw-span on one side of the river, it is best generally to make all of the fixed spans alike. Should each span—because of the gradual shelving off of the bed-rock, and for the sake of economy—be made longer as the bed-rock deepens, the result will be unsightly, even if the increment of span length be regular, for the reason that to an observer there is no apparent motive for thus diversifying the spans.

Any divergence from symmetry and regularity for which there is a self-evident reason produces no unfavorable impression upon the beholder, although it may be sufficient cause for failure to excite his admiration for the structure. If one can see at a glance the *raison d'être* of all the principal parts and peculiar features of a bridge, his sense of fitness will be satisfied, and his general impression will be favorable; but the nearer the approach to perfect symmetry and the more artistic the outlines, the more thorough will be his appreciation of the general effect of the structure.

In making a study of the æsthetics of a bridge design, after determining what spans are applicable, it is well to make one or more layouts on a large scale on the brown paper used in engineers' offices for pencil-drawings, indicating the circumscribing lines of all main members to scale, and tinting or filling between said lines with pencil-shading; then tack the paper on a wall, and stand off at various distances to judge the effect. By doing this one can form a very correct opinion concerning the comparative merits of several layouts, and can ascertain where and how any particular layout can be improved. A consultation with several members of one's office force upon the architectural features of the various designs will often result in an improved effect; for nothing else will bring out both the favorable and unfavorable characteristics of a plan like discussion.

In the outlining of each span a great deal can be accomplished towards beautifying a structure, and there is no better way to study the general effect of any proposed outline than the one just indicated, viz., laying out various trusses to scale, tacking the paper to a wall, and criticising them. It will surprise any one who tries this method to see how quickly he can detect the slightest variation from correctness in outline, and what a difference in effect even a small change in a truss depth will produce. It was in this way that the trusses of the Omaha draw-span were proportioned, and it is doubtful if any improvement could be effected in their outlines when all factors involved in the question are duly considered. In this problem there were but three points to determine, viz., the depths of truss at the two hips and the depth at the tower, for the number of panels was settled by economic considerations, and the straightness and section of the top chords were necessitated by certain questions of efficiency. The depth at the outer hips was first determined by the requirements for clearance, rigidity, and appearance, then the depths at the intermediate hips and tower were settled by trial and discussion from the artistic point of view, due attention being paid to the engineering questions involved by the various inclinations of top chords and inclined inner posts.

In determining the outlines of a span these few elementary principles are to be borne in mind:

1st. There is nothing so ugly in a bridge as parallel chords unless it be a skew. However, for spans between one hundred and twenty-five feet and two hundred feet it is often best to use them, although in certain cases where the loads are great it is practicable to adopt polygonal top chords for spans considerably shorter than the superior limit just mentioned.

2d. While it is generally economical of material to use very long panels, no such extreme length should be adopted as would involve an awkward appearance due to flatness of diagonals.

3d. The curvature of the top chord should be made as great as is consistent with a proper consideration of web stiffness and counterbracing.

4th. When it is practicable in Petit trusses to curve the top chord to such an extent as to make too small the inclination of the end-posts to the horizontal, it is permissible to let the latter extend over one panel only and to make all the main diagonals extend over two panels. The effect is ungraceful, however, when the main diagonals occupy one panel each near the ends of the span, and two panels each elsewhere.

5th. When appearance alone is in question trusses very deep at mid-span are desirable; but an excessive truss depth is conducive to a reversion of bottom-chord stress—a condition which has either to be avoided or provided for by stiffening the bottom chords. In extremely heavy bridges, especially where the dead load is unusually great, it is possible that an undue consideration for economy of metal might cause a designer to adopt a truss depth which would be actually too great for appearance, but this is not likely to occur very often because of other limiting conditions.

6th. There are certain limiting relations between width of bridge, depth of truss, and length of span which for the sake of good effect ought not to be exceeded. Usually the rules established on account of purely engineering questions will prevent these limits from being transgressed, thus proving a maxim which the author has often maintained, viz., that in any design any violation of engineering principles is also a violation of good taste from an artistic point of view.

7th. A very graceful effect can be obtained by placing the lower horizontal struts of the overhead bracing in a cylindrical surface similar to that which contains the panel points of the top chords, but, of course, with different curvature.

In respect to the decoration of each span of a bridge, it may be stated that a little ornamentation is generally much better than a great deal, and that this little should be appropriate and in keeping with the general character of the structure. A prodigal use of cheap cast-iron trimmings at a portal of a steel bridge is not in good taste; but it is perfectly proper to decorate the intersections of the members of the portal bracing by plates or rosettes, to surmount the upper horizontal portal strut by an æsthetically designed parapet, to use ornamental corner brackets beneath the lower portal strut, to employ fancy name-plates symmetrically arranged, and to place ornamental figures of proper size and design at the hips, pedestals, or middle of inclined end-posts. It is also permissible to ornament the intermediate transverse vertical bracing to a slight degree by rosettes and knee-braces, but such decoration should be applied sparingly. Again, in large bridges it is proper to be somewhat extravagant in the use of metal at the portal for the sake of appearance, especially as such metal, if it does not add to the strength of the bridge, certainly increases its rigidity.

The ornamentation of viaducts and elevated railways is something which has never received in America any attention worth mentioning, as is proved by the inherent ugliness of nearly all the elevated roads of our great cities, and the painful plainness of our railway trestles throughout the country. It is principally this neglect of æsthetics in design which has created such bitter opposition on the part of the property owners to the building of elevated roads in the heart of the city of Chicago.

Electric lights and gas-fixtures of artistic pattern can be made great aids in securing a pleasing effect in designs for bridges and viaducts; and at night a well-studied distribution of incandescent lights can be made to produce a brilliant appearance at the portals of any large and important city bridge.

Ornamental hand-rails are also of great service in decorating trestles and bridges, especially in deck structures, where these rails can be built in the form of a highly ornamental parapet.

Architectural effect in bridge-building seldom derives much aid from paint, for the reason that it is generally best, on ac count of both convenience and good taste, to use but one color in painting a bridge. A proper choice of color, however, is a material advantage; and it is correct to vary the color in certain accessory portions of the structure, such as machinery-houses, the lettering on name-plates, etc. Some engineers have advocated painting the tension and compression members of different colors, but this would get one into difficulties in spans where certain strictly tension-members are made stiff. Ornamental figures should be painted of the same color as the rest of the bridge. In general, it may be stated that for ordinary conditions of landscape the heavier the structure the lighter should be the color of the paint used, for the reason that if a bridge has an appearance inclining toward clumsiness this objectionable effect can be lessened by reducing the prominence of its members; while, on the other hand, a bridge which is of such an extremely light and airy design as to produce an appearance of weakness can be made to look stronger by adopting a paint of dark color, and thus bringing its members into greater relief in respect to surrounding objects. With very dark backgrounds, however, it will often be advisable to use a light-colored paint even for slight structures, so as to give the bridge a definite outline.

In regard to the ornamentation of bridges by the adoption of elaborately artistic approaches, but little has yet been done in America, the reason being that any money so expended has evidently no utilitarian purpose, and consequently to the eye of the solely practical man appears to be entirely wasted. In Europe it is customary to ornament large and important bridges in this way; and the time is coming when it will be the practice in America also.

A proper proportioning of piers and abutments has a great deal to do with the obtaining of an artistically designed bridge; but, unfortunately, in these, even more than in the superstructure, the almighty dollar is generally the ruling influence in the design. In many bridges the piers do not seem to be massive enough for the spans; and, as will be shown in Chapter XXII, too often they are not sufficiently large to meet certain important engineering requirements, which are, as a rule, ignored by the average designer, and occasionally even by some who consider themselves bridge experts. In the author's opinion, if piers and abutments be adequately designed from an engineering point of view, they will not fall far short of the ideal of artistic excellence.

In concluding this chapter the author would advise each of his readers to study carefully Chapter XXVI on "The Æsthetic Design of Bridges," by David A. Molitor, Esq., C.E., in Prof. Johnson's work on the "Theory and Practice of Modern Framed Structures." Although most of Mr. Molitor's illustrations are necessarily drawn from European practice, there are many features thereof which it would be well for American bridge-designers to adopt; notwithstanding the facts that European and American practice in bridge-building are fundamentally and essentially different, and that American engineers have little or nothing to learn from their brethren across the seas concern the science of bridge design. From an artistic point of view, however, it must be confessed that the average American bridge is inferior to the average European structure; so, while it is advisable that American bridge-designers study carefully European practice in respect to æsthetics, they should be cautious to avoid thoughtless imitation; because decorative features which are appropriate to the heavy, massive, and costly bridges of Europe would be out of place when engrafted on the light, airy, and economic structures that are characteristic of American engineering practice.

CHAPTER V.

CANTILEVER BRIDGES.

THERE seems to be a notion prevalent among the uninitiated (engineers too often included) that there is some inherent virtue in cantilever bridges which renders them superior to ordinary structures; in what particulars, however, the said uninitiated are not often able to state, although they generally claim that it is in economy.

This notion is entirely erroneous; for cantilever bridges are always inferior in rigidity to bridges of simple truss spans, and, excepting for certain peculiar conditions, are also always more expensive. These exceptional conditions are but two, viz., deep gorges to be crossed by single spans, and the impracticability of using false work because of danger from washout.

If there be assumed a river crossing of very great length, n which the bed-rock is approximately horizontal and where the conditions affecting erection are not unusually dangerous, there is no possible layout for a cantilever bridge which will be as inexpensive as a structure consisting of simple truss spans of equal length, provided that the said length be the most economic one possible. That this fact is not generally known is proved by the occasional building of a cantilever bridge in a place where the conditions do not call for one. For instance, there was no good reason whatsoever for making the great Poughkeepsie Bridge a cantilever structure, because by using the same number of piers and making all the spans alike the cost of the substructure would not have been at all increased, but probably diminished, while the weight of metal in the superstructure and towers would have been lessened materially. It is true there may have been a little saving in cost of false work, but as the materials could have been used several times, it could not have been large; while to partially offset it there is the extra cost of the adjusting apparatus and the greater cost of erection due to delays in making the central connections. Moreover, alternate simple spans could have been erected without falsework by the expedient adopted by the author for several Japanese bridges, which expedient will be described subsequently in this chapter.

There is a small cantilever bridge in Philadelphia close to the Pennsylvania Railroad where it approaches the depot, which as a cantilever has absolutely no *raison d'être*. It makes the observer think that, before it was built, some of the city fathers felt that Philadelphia would be behind the times if she did not have a cantilever bridge of some kind or other, and that they erected this one in consequence.

Other illustrations of unnecessary cantilevers could be quoted, but it would be a useless task to carry the illustration farther.

If a deep, narrow gorge with rocky sides has to be bridged, the cantilever construction will often prove economical for two reasons: first, the main piers, being small, are comparatively inexpensive; and, second, the cost of falsework will be almost entirely eliminated, only a small amount thereof being used for erecting the anchor arms.

Again, if a stream is to be bridged where it is impossible to put in falsework, or where there would be danger of its being washed out in case it could be put in, the cantilever will prove an economic design, although in certain cases the cantilever arch design described in Chapter VI. may be still more economical and possibly more rigid. This last feature, however, will depend somewhat upon the character of the arch adopted.

That a cantilever bridge is less rigid and deflects more vertically than a simple span bridge, no one who has examined both types of structure under load and who has measured the vertical deflections can well deny; nevertheless this comparative lack of rigidity is no great detriment or weakness, and should not be allowed to militate against the building of a properly designed cantilever bridge where the conditions call for such a structure. Compared with a suspension bridge, a cantilever bridge is rigidity itself. But, again, this is no reason for condemning *in toto* suspension bridges, which have their legitimate place in engineering construction, viz., where either an extremely long span is necessary, or where a cheap highway bridge over a wide river is required.

There is but one kind of steel structure in which the cantilever is more economical of metal than the simple span, viz., roofs supported on steel columns, as in train-sheds and workshops. The reason for this economy is the shortening of the spans and the ignoring of the effects of reversion of stress when proportioning members. The latter is legitimate within certain limits because of the infrequency or improbability of such reversion.

Cantilever bridges being of such an unusual type, and their use with very few exceptions dating back only about twenty years, but little effort has yet been made to systematize their designing or to investigate their economic features. The only paper of any real value on the subject, which has come to the author's notice, is one by Prof. Edgar Marburg, published in the *Proceedings* of the Engineers' Club of Philadelphia for July, 1896. This paper is an excellent one, but it really does not settle any important point concerning the economic relations of span lengths, for its mathematical investigations are rather crude approximations.

As the author has lately in his practice accumulated a mass of data concerning weights of metal in cantilever bridges, he has had his assistant engineer, Mr. Hedrick, extend his calculations not only so as to determine all the economic relations of cantilever bridges, but also so as to prepare percentage curves, by using which the total weight of metal in any cantilever bridge of any ordinary type can be found very quickly and with considerable accuracy.

Before proceeding to present these results, though, several other matters will receive consideration.

In no work on bridges, that the author has ever seen, has there been given a statement of the various stresses for which the several spans of a cantilever bridge should be figured; so such a tabulation is herewith presented.

STRESSES IN SUSPENDED SPAN.

First. Dead-load Stresses.
Second. Live-load Stresses.
Third. Impact-load Stresses.
Fourth. Direct Wind-load Stresses.
Fifth. Transferred-load Stresses.
Sixth. Erection Stresses from Dead Load.
Seventh. Erection Stresses from Wind Load.

STRESSES IN CANTILEVER-ARMS.

First. Stresses due to Dead Load on Suspended Span.
Second. Stresses due to Live Load on Suspended Span.
Third. Stresses due to Impact Load on Suspended Span.
Fourth. Stresses due to Wind Load on Suspended Span.
Fifth. Stresses due to Transferred Load on Suspended Span.
Sixth. Stresses due to Erection of Suspended Span and caused by the Dead Load.
Seventh. Stresses due to Erection of Suspended Span and caused by the Wind Load.
Eighth. Stresses due to Dead Load on Cantilever-arm.
Ninth. Stresses due to Live Load on Cantilever-arm.
Tenth. Stresses due to Impact Load on Cantilever-arm.
Eleventh. Stresses due to Wind Load on Cantilever-arm
Twelfth. Stresses due to Transferred Load on Cantilever-arm.

This load affects only the main inclined posts over piers.

STRESSES IN ANCHOR-ARMS.

First. Stresses due to Dead Load on Suspended Span.
Second. Stresses due to Live Load on Suspended Span.
Third. Stresses due to Impact Load on Suspended Span.
Fourth. Stresses due to Wind Load on Suspended Span.
Fifth. Stresses due to Transferred Load on Suspended Span.

Sixth. Stresses due to Erection of Suspended Span and caused by the Dead Load.

Seventh. Stresses due to Erection of Suspended Span and caused by the Wind Load.

Eighth Stresses due to Dead Load on Cantilever-arm.

Ninth. Stresses due to Live Load on Cantilever-arm.

Tenth. Stresses due to Impact Load on Cantilever-arm.

Eleventh. Stresses due to Wind Load on Cantilever-arm.

Twelfth. Stresses due to Dead Load on Anchor-arm.

Thirteenth. Stresses due to Live Load on Anchor-arm.

Fourteenth. Stresses due to Impact Load on Anchor-arm.

Fifteenth. Stresses due to Wind Load on Anchor-arm.

Sixteenth. Stresses due to Transferred Load on Anchor-arm.

Stresses in Main Central Spans.

CHORD STRESSES.

First. Stresses due to Dead Load from both Suspended Spans and Adjacent Cantilever-arms.

Second. Stresses due to Live Load covering both Suspended Spans and Adjacent Cantilever-arms.

Third. Stresses due to Impact for the latter case.

Fourth. Stresses due to Wind Load on both Suspended Spans and both Adjacent Cantilever-arms.

Fifth. Stresses due to Transferred Load on both Suspended Spans.

Sixth. Stresses due to Dead Load on Main Central Span.

Seventh. Stresses due to Live Load on Main Central Span.

Eighth. Stresses due to Impact Load on Main Central Span.

Ninth. Stresses due to Wind Load on Main Central Span.

Tenth. Stresses due to Transferred Load on Main Central Span.

Web Stresses.

First. Stresses due to Dead Load on both Suspended Spans and both Cantilever-arms.

These will be zero for a symmetrical structure.

Second. Stresses due to Live Load on one Cantilever-arm and one adjoining Suspended Span.

This loading produces a constant shear from end to end of Main Central Span.

Third. Stresses due to Impact from last load.

Fourth. Stresses due to Transferred Load on one Suspended Span.

This loading produces a constant shear from end to end of Main Central Span.

Fifth. Stresses due to Dead Load on Main Central Span.

Sixth. Stresses due to Advancing Live Load on Main Central Span

Seventh. Stresses due to Impact from last load.

For certain conditions some of these stresses will not need to be considered, but in other cases they will, consequently it is necessary to insert them in the lists. For instance, in the cantilever and anchor arms the sixth and seventh items will generally be found to have no influence on the sections of members, but in some cases they will, as in long-span highway bridges with light live loads.

In calculating erection stresses, the weight of the traveller must not be forgotten, as its influence on such stresses is by no means inconsiderable.

The combination of the various stresses requires both judgment and care, for some loads may or may not act together, and some produce tension while others produce compression in the same member. Again, distinction must be made between groups of stresses with and those without wind-stresses, so as to use the different intensities of working-stresses given in the specifications of Chapters XIV. and XVI. It would be too tedious to give here the various combinations of stresses for each member of each span; but it will suffice to say that the computer will have to find for each main member in the entire bridge the greatest tension when wind-stresses are included, the greatest tension when they are excluded, the greatest compression when they are included, and the greatest compression when they are excluded, taking care not to group together any stresses that cannot exist simultaneously.

The determination of the proper live load per lineal foot for any member of a cantilever bridge is one requiring a little care, the rule being that for the piece considered the length of span to be used in applying the live-load diagram is the total length of structure which must be covered by the moving load in order to obtain the greatest stress in the said piece, excepting only the suspended span and the main central span, for which the live loads actually imposed are to be treated exactly like those of simple spans. Of course, the impact is to be figured for the length of structure that must be covered by the live load to produce the greatest stress in the piece under consideration.

Some young engineers have an idea that the finding of stresses in cantilever bridges is a complicated matter. On the contrary, it is very simple, as every stress can be determined by the ordinary principles of statics and very readily by the use of graphics. Although the work is simple, it is somewhat long and tedious, as is evident from the preceding lists of stresses. The computer is advised, when finding the stresses, not to try to group the loadings any more than they are grouped in the said lists, for, if he does, he will probably have to separate them while making his combinations.

In respect to combinations of stresses during erection, there will be no necessity for increasing the sections proportioned for other combinations, provided they are as large as those required by the said erection-stress combinations with the intensities given in the specifications (Chapter XIV.) for combinations that include wind-stresses, viz., intensities thirty per cent higher than those for combinations without wind stresses.

Cantilever bridges may be made either through, deck, or half-through; but a combination of deck-spans for the anchor-arms and a through-span for the cantilever-arms and suspended span is awkward-looking and unsightly. There is a structure of this type across the St. Lawrence River, near Montreal.

It is no easy matter to give an artistic effect to a cantilever bridge; nevertheless it is generally within the realms of possibility to do so, although it must be confessed that most of the existing structures of this type are uncompromisingly tingly. If a convex upward curve can be placed in the top chord of the suspended span, so as to reverse at the ends into a concave upward curve on the cantilever-arm, a graceful effect will be obtained; but the design generally will not be economical for erection on account of the large erection-stresses near the point of suspension. The author has made a design on these lines for a proposed 1500-ft. span highway bridge to cross the Mississippi River at St. Louis; and, as the suspended span would be erected on falsework, there is no want of economy involved. The layout with all the main members drawn to true scale has a very pleasing effect.

In long spans like this it becomes necessary to widen the cantilever and anchor arms uniformly from ends to main piers, so as to obtain the requisite rigidity for resisting wind-pressure and so as to keep the wind-stresses in bottom chords within reasonable limits. It seldom pays, however, to build the trusses of these arms in planes inclined to the vertical, principally because of the complicated shop-work involved.

The author has lately had occasion to design a number of large bridges for a proposed branch-line of the Nippon Railway of Japan. The line, which will be about one hundred miles long, is to follow the course of a mountain torrent that rises from twenty to twenty-five feet in two or three hours, and attains in places a depth of water exceeding one hundred feet with a total rise of sixty feet. Of course, falsework can be employed for these bridges only to a very limited extent, hence it was necessary to resort to the use of the cantilever. Three of the eight structures were designed as ordinary cantilevers, two as simple truss-bridges, and three as cantilevers during erection and simple spans afterwards. The last style of bridge is very economical of both metal and money, and will bear further investigation and extension, so as to be made applicable to crossings where the ordinary cantilever bridge would otherwise be adopted. Its *modus operandi* is as follows:

At each side of the river there is erected on false work a simple span having its chords and certain of its web-members (or for short spans all of them) stiffened for erection-stresses. Then over each pier is built a toggle consisting of horizontal upper-chord eyebars and adjustable verticals, by means of which one half of the central span is cantilevered over the stream to meet the other half, after which the toggles are to be removed. This method of erection can be understood by reference to the diagram in Fig. 1.

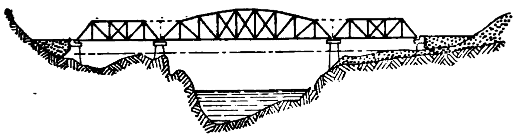


FIG. 1.

One of the three cases mentioned had rather peculiar conditions, which necessitated the adoption of another expedient. About midstream there is a narrow rocky island that reaches to about the elevation of extreme high water. Near the edges of this island, as shown in Fig. 2, will be built two small piers,

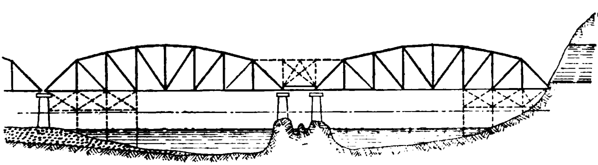


FIG. 2.

each of which will support one end of a long span. Between the end shoes will run a temporary strut, and from each pedestal will spring a temporary post to support the temporary top-chord eye-bars that run from hip to hip. The rectangular panel is braced with temporary adjustable diagonals, and the top chord is hinged at the middle and connected to the pedestals by other temporary adjustable rods. These two sets of adjustable rods permit of the raising or lowering of one span at a time. By means of this device more than one half of each span can be cantilevered out to meet the remainder thereof, which will be erected on falsework.

It is intended to erect the cantilevered portions of all three bridges with their ends higher than they will be in their final position, so that no raising, but only lowering, of the weight of the arms by the toggles will be necessary. The author is of the opinion that these toggles will work much more easily, and will prove in the end less costly, than the wedges used for adjustment in the erection of the Red Rock Cantilever Bridge, a description of which was given by Samuel M. Rowe, M. Am. Soc. C. E., in the *Transactions* of that Society for 1891.

In one of the three true cantilever bridges for the proposed Japanese railroad an expedient has been adopted by the author which may be worthy of description. One approach to the structure, as shown in Fig. 3, is through a tunnel end-

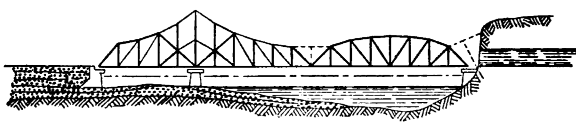


FIG. 3.

ing in the face of a vertical wall of rock. It was at first intended to use this rock in lieu of one anchor-arm of an ordinary cantilever by letting the main posts lie close to its vertical face and tying the top chords well back into its mass; but a study of the contours of the rock showed that it dipped off to one side of the line in such a way as to render such an anchorage of uncertain strength, so it was decided to increase the lengths of the suspended span and far cantilever-arm sufficiently to cut out the near cantilever-arm, and thus let the end of the suspended span roll on two small pedestals at the mouth of the tunnel. Five eighths of this span will be erected by toggles fastened into the rock, and the remaining three eighths will be cantilevered out also by toggles from the end of the far cantilever-arm. This method requires more metal than does the one first contemplated; nevertheless it is the cheapest, everything considered, that can be adopted. The rock-anchorage is amply strong for the dead-load pulls on it during erection, although, as before stated, it is not sufficiently reliable for resisting the effects of live loads.

The best method of attaching the suspended span of an ordinary cantilever bridge is by hangers from inclined end posts on the cantilever-arms. For such suspenders narrow eye-bars should be used; and it is generally better to hinge them at the middle. This is because they are subjected to transverse bending, due to longitudinal expansion and contraction of suspended span from both changes of temperature and the application and removal of the live load. Narrow bars can spring slightly without being overstrained, and a rotation of the eyes on the pins will thus be prevented. Such a rotation would eventually enlarge the eyes and cut notches into the pins, necessitating for some future time expensive repairs.

A suspended span thus hung is free to move longitudinally under thrust of train, but its ends are tightly held in a lateral direction, so that all wind loads are carried properly to the bottom chords of the cantilever-arms; and excessive longitudinal motion is prevented by the continuity of the track.

In cantilever-arms it is better and more economical to use inclined posts as well as vertical ones over the piers, so that the various loads will be carried more directly to the masonry. To insure the travel of the wind stresses down the transverse bracing between these inclined posts, instead of up to the apex of the top chord and down the bracing between the vertical posts, the author leaves out one pair of diagonals of the upper lateral system between the said apex and the tops of the inclined posts. The same expedient is used also for the anchor-arms and between the hips of the suspended span and the cantilever-arms.

All bracing between opposite vertical posts and between opposite inclined posts should be made very rigid; and in double-track structures all the sway-bracing should be proportioned to carry as a live load, with the proper allowance for impact, the greatest shear which can come upon it from loading one side of the floor only.

Great care is necessary in designing the pedestals over the main piers so as to carry the loads from the three heavy posts to the masonry without overstraining any of the metal in the pedestal, and so as to distribute the total pressure uniformly over the masonry bearing. Until recently the author has adopted pedestals built of plates and shapes, but has lately decided to try steel castings, as the pound price for these has now come down to something like a reasonable figure. The difficulty in finding room for the proper number of rivets for attaching together their component parts renders built pedestals clumsy and expensive.

The anchorage details require special care, and no rules can be given to govern their designing, for the reason that the conditions vary for all crossings. The following hints, though, may be of use to the designer :

First. The anchor-bars should be made as long and as narrow as practicable, and should be divided into short lengths by pins, for the same reason as given in the case of the suspenders of the suspended span.

Second. All anchorage details should be accessible to the paint-brush, excepting, of course, those portions of the bottom girders which are buried in the masonry. This result is accomplished by leaving wells in the anchorages of sufficient size to permit the passage of a man to do the painting. If these wells are at any time partially filled with water temporarily by the rise of the stream, no harm will be done, provided that the painting of the metal-work therein be always attended to properly.

Third. Concrete for anchorages is always preferable to masonry, because it can readily be made to take any required form. If necessary, its exterior can be protected against abrasion from ice or drift by facing with granite or other hard rock.

Fourth. There should be an independent anchorage against wind-pressure, obtained by sliding surfaces of steel, one of each pair of same forming part of a heavy detail which is rigidly attached to the bottom of the end floor-beam, and the other forming part of a heavy detail that is anchored firmly to the masonry.

Fifth. The tops of the anchor-piers should be made absolutely water-tight without interfering with the longitudinal expansion of the anchor-arm, so as to prevent rusting of the interior metal-work.

Sixth. The net weight of masonry in any anchor-pier, after deducting the greatest buoyant effort of the displaced water, should be twice as great as the maximum uplift on the said anchor-pier, when the effect of impact is duly included.

A few observations concerning some of the largest cantilever bridges yet built may be of service to the reader:

The largest structure of this type in the world is the bridge at Queensferry over the Firth of Forth, the main portion of which consists of two spans of 1710 ft. each, with central spans of 350 ft. each, and two anchor-arms of 680 ft. each. The length of the tower-span over the centre pier is 260 ft., and that of each of the two other tower-spans is 145 ft., making the total length of the main structure 5410 ft. The design for this bridge and a complete history of its construction are given in a special work published by *Engineering* (London).

The exceptions which the author would take to this design are as follows:

First. The suspended spans are just about one half as long as they ought to be for both appearance and economy.

Second. The structure should have been made pin-connected for both ease of erection and certainty of stress distribution.

Third. A single system of cancellation for the webs of the girders would have been more scientific than the double system adopted, and would not have been any more expensive.

Fourth. The structure as a whole, from the point of view of American engineers, was unnecessarily expensive.

On the other hand, though, the labor involved in both the designing and building of this bridge was immense; and the successful completion of the structure is a great credit to all concerned in its designing and construction.

The cantilever bridge having the next longest span is the Lansdowne Bridge over the Indus River at Sukkur, India.

It consists of a single span of 820 ft. without anchor-arms, the latter being replaced by guys, and with a suspended span of 200 ft. The appearance of the bridge is bizarre in the extreme, and the structure is economic in neither weight of material nor cost of shop-work. Compared with an American bridge of the same span, capacity, and strength, the weights of metal in the 820-ft. span only would be about in the ratio of unity to 0.75.

The cantilever having the next longest span, viz., 790 ft., is the railway bridge at Memphis over the Mississippi River. This structure is both unsightly and uneconomical of material. Its layout of spans is unfortunate (but the War Department, and not the designer, is responsible for this), and the truss depths are far too small for both economy and appearance.

The next longest cantilever span is that of the Red Rock Bridge over the Colorado River on the Atlantic and Pacific Railway. This structure consists of a main span of 660 ft. and two anchor-arms of 165 ft. each, the length of the suspended span being 330 ft. The width between central planes of trusses is 25 ft.; and the truss depth varies from 55 ft. for the suspended span to 101 ft. for the vertical posts over the main piers. As the author is the person responsible for its layout, his criticism thereof will not be of much value. The bridge was designed to meet certain conditions, economy in first cost being the prime requisite; consequently the subject of æsthetics did not receive great consideration. Engineers and architects differ fundamentally in their opinions concerning the architectural effect in this structure. Some approve its appearance; others characterize it as harsh in its outlines. The relations between lengths of suspended span, cantilever-arms, and anchor-arms, and those of width and depth, although very hurriedly determined, have since been found to be just about the best practicable. This bridge, as before stated, is described very fully in the *Transactions* of the American Society of Civil Engineers for 1891.

There are many other cantilever bridges having main spans of from 400 ft. to 500 ft. or more, but space will not permit their enumeration.

Many expedients have been used to connect the metal-work of the suspended spans of cantilever bridges, and considerable trouble has often been experienced in doing the work, owing to variations in both length and elevation. The author is of the opinion that but little difficulty will be experienced if the following precautions be taken:

First. See that the entire triangulation is so accurately done that there will be no possibility of an error exceeding one quarter of an inch in the distance between centres of pins over main piers. A perusal of Chapter XXIII. will show that this is perfectly feasible.

Second. See that extra precautions are taken by the inspectors during the manufacture of the metal-work to insure that all lengths of main members shall be absolutely correct.

Third. See that the tapes used in shop and field are of exactly the same length.

Fourth. Use toggles like those described in this chapter for effecting the adjustment.

Fifth. Arrange to have the meeting ends of the chords a trifle high, so that lowering and not raising will be necessary.

Sixth. Arrange matters so that when the ends of the metal-work come together they will be a trifle apart rather than tending to lap, for it is much easier to heat the chords slightly by suspending beneath them sheets of metal containing slow fires than it would be to cool them by packing ice around them in cloths.

Referring now to the before-mentioned special investigations made by Mr. Hedrick, the questions set him for solution at the outset thereof were the following:

First. The ratio of the economic length of suspended span to that of the total opening.

Second. The most economic length of anchor-arms when the total length between centres of anchorages is given, and when the main piers can be placed wherever desired.

Third. The relations between the weights of metal in the suspended span, cantilever-arms, anchor-arms, anchorages, main pedestals, and anchor-spans.

Fourth. The best proportionate length for anchor-spans, and the comparative weights of metal in those of different lengths.

Fifth. The ratio of weights of metal in cantilever bridges of various types to those in simple-span bridges having the same number of spans.

Mr. Hedrick's method of determining the economic functions was to take the data on hand for the proposed Japanese cantilever bridges, exact weights of metal having been computed for structures of 320-ft., 400-ft., and 500-ft. openings, and, by varying the layouts so as to use longer and shorter suspended spans and longer and shorter anchor-arms, obtain, by actual designs and estimates, the weights of metal for a sufficient number of layouts to indicate the desired minima.

In determining the economic length of suspended span for a certain opening, the length for the anchor-arms was first assumed to be one fourth of said opening, then the total weight of metal in the entire bridge, including even the anchorages and pedestals, was figured for several cases; and the length of suspended span giving the least weight of metal for the whole structure was found to be about three eighths of the opening, although this length showed only one and a half per cent advantage over the case where the ratio was one half. Now, as the rigidity of the entire structure certainly increases with the length of the suspended span, it will often be found best to make the length of the latter about one half of the opening rather than three eighths or any smaller proportion. On the other hand, though, it has been found by trial that, with the three-eighths ratio, there results a more sightly layout than can be obtained with the one-half ratio.

Next Mr. Hedrick tabulated the various component truss and lateral weights of several of the typical cantilever bridges designed in the author's office, the leading dimensions for which are given in the following table.

From these weights he constructed the curves shown on Plate X, from which can be found the total weight of metal in the trusses and lateral systems of any three-span cantilever bridge, when the weight per lineal foot of the trusses and laterals in the suspended span is known. This weight, by the way, is, on the average, eight per cent greater than that for an ordinary simple span of the same length, the extra metal

TABLE OF DATA FOR CANTILEVER BRIDGES USED IN DETERMINING CURVES OF WEIGHTS.

Name of Structure.	Length of Central Opening.	Length of Suspended Span.	Length of Cantilever-arms.	Length of Anchor-arms.	Truss Depth of Suspended Span.	Height of Towers.	Panel Length.	C. to C. of Trusses.
Agano Gawa Crossing No. 4..	320′	120′	100′	100′	22′	49′	20′	16′
Agano Gowa Crossing No. 7..	400′	200′	100′	100′	28′	62′	25′	18′
Agano Gawa Crossing No. 5B	500′	200′	150′	125′	40′ at centre 28′ at hips	73′	25′	20′
Red-rock Cantilever	660′	330′	165′	165′	55′	101′	27′ 6″	25′
Red-rock Cantilever, alternative design.........	830′	500′	165′	165′	69′ at centre 53′ at hips	111′	31′ 3″	30′
St. Louis Highway...........	1500′	760′	370′	370′	95′ at centre 70′ at hips	195′	35′ for susp. span 37′ for arms	38′ for susp. span 60′ at towers

being required mainly for stiffening certain truss members to resist erection stresses. Of course, if false work be used for the suspended span, the eight per cent excess will not be added.

The curves of percentages are based on two assumptions, viz.: first, the panels throughout the entire structure are of equal length, and, second, the lengths of the cantilever-arms and anchor-arms are the same. The first assumption is nearly always correct, for there is no advantage to be gained by varying the panel lengths in the various portions of the bridge. If the lengths of cantilever and anchor arms are unequal, the average weight of metal obtained for the latter by use of the curve will have to be corrected by the formula

$$T' = \frac{T}{2}(1 + r),$$

where T'' is the correct, final weight of truss and lateral metal in the anchor-arm, T is the weight of same found by the percentage curve, and r is the ratio of length of cantilever-arm to that of anchor-arm.

It should be observed that, in applying the percentage curves to structures having subdivided panels like those of the Petit truss, the main or double panel is to be used as the basis of calculation.

The method of applying the percentage curves is as follows: Let us take any opening and assume that there are six panels in each cantilever-arm, and that the weight per foot of truss and lateral metal in the suspended span is w, the panel length being p, and $pw = W$. It is to be observed that this method is applicable for any proportionate length of suspended span.

The weight of metal in the floor system, being independent of the span length and simply a function of the panel length and of the distance between trusses, is not considered in the investigation, but is, of course, to be added when figuring the total weight of metal in the structure.

The weight of truss and lateral metal in the cantilever-arm will be

$$1.2W + 1.4W + 1.65W + 2.0W + 2.4W + 3.0W = 11.65W.$$

The weight of metal in the panel over the pier is, according to the directions on the diagram,

$$1.8 \times 3.0\,W = 5.4\,W.$$

Let us assume that there are only five panels in the anchor-arm, then the trial weight T will be

$$0.75\,W + 1.75\,W + 2.1\,W + 2.5\,W + 3.0\,W = 10.10\,W.$$

Substituting in the formula gives

$$T'' = \frac{10.10\,W}{2}\left\{1 + \frac{6}{5}\right\} = 11.11\,W.$$

It will be seen from these calculations that the full percentages given for the end panel points of cantilever and anchor arms are to be used, although in reality there is but a half panel length for each point. This is caused by the heavy details required at these points for adjustment and anchorage. All erection metal at the end of a suspended span is assumed to belong to the cantilever arm.

Should in any case the panel lengths be unequal in different portions of the structure, it will be a simple matter to use the curves by finding average weights per foot for two assumed cases of equal panel lengths, one making the arm greater and the other making it less in length than it actually is, and interpolating properly between the results for the required average weight per foot for the arm.

The total weight of metal in the two anchorages of any three-span cantilever bridge can be taken at five per cent of the grand total weight of metal in the said three spans, and the weight of metal in the pedestals on main piers at four per cent of same. Of course, conditions vary for different cases, nevertheless these percentages will give results sufficiently close for all practical purposes.

If the bridge be so long as to require an anchor-span, its weight of truss and lateral metal per lineal foot will be about $3.25w$, irrespective, strange to say, of the length of said anchor-span, w being the weight per foot of the trusses and laterals in a suspended span, whose length is three eighths of the main opening. The explanation of this is that the weight per foot of the chords, though independent of the upward bending moment, increases proportionately to the downward bending moment with the length of span; while the weight per foot of the web, in so far as it is affected by the shears from exterior loading, the ruling factor in determining the sections of web members, varies inversely as the span length.

If the length of anchor-span be very short, say materially less than one half of the main opening, the weight per foot for trusses and laterals will have to be increased to $3.5w$, notwithstanding the fact that the entire top chords may then be built of eye-bars; but such short spans would probably be barred out by consideration for navigation interests.

The percentage curves of Plate X will not bear a rigid criticism, in that they make the weight of metal depend upon the number of panels. It is presupposed, however, that the panel length adopted is the most appropriate one for the bridge; and the curves will be found quite accurate whenever the proper panel length is used. With long panels the weight of metal per lineal foot found by the curves for cantilever and anchor arms is less than that found thereby for short panels. This is as it should be, but to a limited extent only; for it can be found by trial that an abnormally short or abnormally long panel length will give results too great or too small when checked by computations of weights made from actual designs.

These percentage curves enabled Mr. Hedrick to solve readily the next problem, viz., given the total distance between centres of anchorages and *carte blanche* as to the location of the main piers, to determine the length of each anchor-arm which will make the total weight of metal in the structure a minimum. He found this length to be two tenths of the total distance between the anchorages.

It must not be forgotten that for every dollar saved by reducing the total weight of metal through the shortening of the anchor-arms, it will be necessary to spend about twenty cents for extra concrete in the anchorages. On this account, for the conditions assumed, the truly economic length of each anchor-arm of a three-span cantilever will generally be a little greater than twenty per cent of the total distance between centres of anchorages.

When, however, the problem is to determine the economic length of anchor-arm for a fixed distance between main piers, the result will be quite different; because, within reasonable limits, the shorter the anchor-arm the smaller will be its total weight of metal, and because trestle approach is much less expensive than anchor-arm. It would not, for evident reasons, be advisable to make the length of anchor-arm less than twenty per cent of that of the main opening, or say fifteen per cent of the total distance between centres of anchorages. With this length there would probably be no reversion of stress in the chords of the anchor-arm, even when impact is considered. Generally, though, the appearance of the structure will be improved by using longer anchor-arms than the inferior limit just suggested.

In respect to the best proportionate length of anchor-spans, the latter weigh so much per lineal foot for all cases that the shorter they are made the greater the economy; but, as before stated, it is improbable that navigation interests would ever permit of their being made shorter than one half of the main openings.

In respect to his fifth and last problem, Mr. Hedrick obtained the following results:

The total weight of metal in a three-span cantilever railroad bridge, floor system included, is to the total weight of metal in a simple-span bridge of three equal openings, for which false work is to be used throughout, as unity is to 0.6. The corresponding ratio for the case of the centre span, erected without false work, is unity to 0.64.

For a very long bridge, composed of a succession of cantilevers and anchor-spans which are one half as long as the main openings, and which has a suspended span resting on each extreme pier, the ratio of weight of metal to that in a corresponding bridge of equal, simple spans and the same number of piers, the spans being erected on false work, is as unity to 0.75. For the case of alternate simple spans erected without false work, the ratio would be as unity to 0.8. These results were obtained by assuming average probable conditions; but the longer the simple spans and the greater the total length of structure, the less will be the variation in weights of cantilever and simple-span bridges, although it would require very long spans and a great total length of structure to change materially the ratios found.

It is therefore evident that, when economy in first cost is considered, as it always ought to be, there will seldom, if ever, be any need for considering the adoption of cantilever bridges with anchor-spans, because structures with simple spans are both cheaper and better. It is also evident that in many cases it is advisable, from considerations of both rigidity and economy, to adopt a bridge consisting of three simple spans, with the middle one cantilevered from the others, rather than the ordinary three-span cantilever bridge. When each of the side spans is as short as one-half of the middle span, or even shorter, there will be no difficulty experienced in the erection, and no great provision will be required for holding down the outer ends of the side spans during erection. Of course, the nearer to equality that the three span lengths are made, the greater will be the economy of metal, but a wide divergence in these lengths will not necessitate any such increase in weight as to alter the preceding conclusion regarding the great economy of simple-span bridges over ordinary cantilever structures.

The question sometimes arises as to how the total weight of metal in a three-span cantilever bridge varies with the length of the main opening. If the lengths of the anchor-arms vary proportionately to the main opening, the increase or decrease in the total weight of metal in the structure will vary about twice as rapidly as the increase or decrease in length. For instance, if the main opening and total length of bridge be increased ten per cent, the total weight of metal in the entire structure will be increased twenty per cent. This rule, which is merely an approximation, will apply fairly well for changes not exceeding twenty per cent and for spans of medium length. For greater changes the ratio of increase or decrease gradually augments, and for very long spans it is slightly greater than two, while for very short ones it is slightly less.

In respect to the relations which should exist between length of main opening, perpendicular distance between central planes of trusses, and the various truss depths, the author's practice is to make the least distance between parallel trusses one twenty-seventh of the main opening; the least distance between axes of vertical posts over main piers, when the trusses converge towards the suspended span, one twenty-fifth of the said opening; the truss depth for the suspended span, when the chords are parallel, from one fifth to one sixth, or for very long spans even one seventh, of the span; and the height of the vertical posts over main piers not to exceed four, or preferably three and a half, times the perpendicular distance between their axes. For through cantilever bridges the author generally makes the height of these posts about fifteen per cent of the length of the main opening.

For the sake of appearance the centres of the top-chord pins in cantilever-arms are placed on arcs of parabolas, the vertices of which are located at the hips of the suspended span; and the anchor-arms are laid out to the same curve, beginning at the tops of the posts over the main piers.

In concluding this chapter, a check on the correctness of percentage curves for weights of cantilevers will be given by applying the curves to the published estimated weights of metal in the various members of the longest cantilever bridge that has ever yet been designed in detail, viz., the proposed 2300-ft. span (measured between centres of main piers) for the North River Bridge at New York City. This proposed structure was designed by the Union Bridge Company.

The total weight of metal in trusses and laterals of the 720-ft. suspended span is 10,400,000 lbs. The trusses, which are of the Petit type, are divided into six main panels of 120 ft. each; consequently the panel weight is $10,400,000 \div 6 = 1,733,000$ lbs. In the cantilever-arm there are six and five-eighths main panels; consequently the weight of trusses and laterals therefor will be

$$1.20\,W + 1.40\,W + 1.65\,W + 2.00\,W + 2.40\,W + 3.00\,W \\ + \tfrac{5}{8} \times 3.60\,W = 13.90\,W = 24,090,000 \text{ lbs.}$$

Each anchor-arm is 840 ft. long, and is divided into seven double panels, and there are seven and five-eighths loads to be considered; consequently the weight of trusses and laterals therefor will be

$$0.75\,W + 1.75\,W + 2.10\,W + 2.50\,W + 3.00\,W + 3.75\,W + 4.75\,W + \tfrac{5}{8} \times 5.65\,W = 22.13\,W = 38{,}351{,}000 \text{ lbs.}$$

This weight must be reduced, owing to the fact that the length of the cantilever arm is only six sevenths of that of the anchor-arm, making $r = 0.857$.

$$T' = \frac{T}{2}(1 + r) = \frac{38{,}351{,}000}{2}(1.857) = 35{,}609{,}000 \text{ lbs.}$$

The total weight by the curves for the two cantilever and anchor-arms is therefore

$$2(24{,}090{,}000 + 35{,}609{,}000) = 119{,}398{,}000 \text{ lbs.}$$

The total weight of metal given in the published estimate for trusses and laterals for the two cantilever and anchor arms, after deducting 11,500,000 lbs. for weight of metal in the anchorages and ignoring the allowance for sundries (which was probably put in for prudential reasons), is 119,700,000 lbs., making the difference 302,000 lbs., or about one quarter of one per cent.

This is an extremely accurate check, and proves that the curves are reliable; nevertheless the author would not guarantee them to give any such close coincidence for all cases.

Since these pages went to press the author has been engaged on the making of a preliminary design with a detailed estimate of weight of metal for a proposed double-track railway and highway cantilever bridge, with a central opening of 1,600 ft., to cross the St. Lawrence River near Quebec, Canada. The result of the estimate as far as it has been carried gives another excellent check on the accuracy of one of the curves; as the error for the cantilever arms is only one-eighth of one per cent. The anchor arms have not yet been detailed

CHAPTER VI.

ARCHES.

THE arch is a rather uncommon type of structure in America, because the conditions which make it economical are unusual. For deep gorges with rocky sides, or for shallow streams with rock bottom and natural abutments, arches are eminently proper and economical. But when a steel bottom chord is needed to take up the thrust between springing points, all the economy of the arch vanishes.

The advantages of the arch are a possible economy of metal and an æsthetic appearance, while its disadvantages are a lack of rigidity and, for most types, an uncertainty concerning the maximum stresses in the members.

Arches are sometimes used for large train-sheds, in which their architectural effect is certainly very fine, but they require about twice as much metal as do cantilevered trusses supported on columns; consequently they can be adopted only when appearance is an extremely important factor in the design.

When bridge foundations have to be built on piles or on any other material that is liable to slight settlement, or when the abutments could possibly move laterally even a mere trifle, it is not proper to adopt an arch superstructure; for any settlement or any motion whatsoever in either piers or abutments would upset the conditions assumed for the computations, and thus cause to be increased to an uncertain amount some of the stresses for which the superstructure was proportioned. This criticism does not apply to the three-hinged arch, but even this design requires good, solid abutments and firm foundations for piers.

Arches can be erected on false work, by cantilevering, or by building vertically the two halves and lowering them by cables till they meet at the centre. Whichever of these methods is the easiest and cheapest is the one to adopt.

A very easily erected arch is shown in Fig. 4. The pieces marked *AB* are temporary, and are to be used only during erection. They can be made of timber, so as to be removed

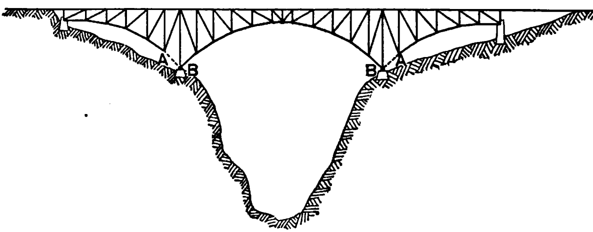


FIG. 4.

readily after the arch is coupled at mid-span, or may be of steel, and be left in as idle members, solely for the sake of appearance.

It will be seen from the diagram that the structure is a cantilever during erection, and afterwards consists of an arch span and two simple spans. This type of bridge probably requires a little more metal than would an ordinary segmental arch with trestle-approaches, and possibly is not quite as rigid as the latter, but the saving of cost in erection will fully offset these disadvantages.

With three hinged arches there is no ambiguity whatsoever in the determination of stresses, but in all other cases there is.

There are four cases all told, viz.:

1. Arch without any hinges.
2. Arch with one hinge (at crown).
3. Arch with two hinges (at abutments).
4. Arch with three hinges (at crown and abutments).

These four cases can be reduced to three, because there is no good reason for ever building an arch fixed at the abutments and hinged at the crown.

In Case No. 4 there are no temperature stresses, but in all of the other cases there are; and they must always receive due consideration in proportioning the members.

All things considered, the author prefers to adopt the three-hinged arch for railroad bridges, because the stresses can be determined as accurately as can those of an ordinary truss bridge, and because of the absence of temperature stresses; at the same time it must be admitted that an arch without hinges is more rigid than one with hinges, and that, theoretically, it is more economical of metal.

For highway bridges, in which the assumed live loads will seldom, if ever, be realized, it would be best, all things considered, to adopt the arch without hinges, so as to obtain the greatest possible rigidity, even at the expense of certainty in computing stresses.

For arched train-sheds, the two-hinged arch of crescent shape will generally be found the most satisfactory.

While the author was engaged on the preparation of this chapter he received a copy of Prof. Malverd A. Howe's new book, entitled "A Treatise on Arches." This work, which is entirely mathematical in character, is certainly the most complete book on arches that has ever been written, and appears to cover the entire subject of stresses in arches of all kinds in a most satisfactory manner, although, of course, the author cannot vouch for the correctness of Prof. Howe's figures without checking the mathematics from start to finish, a task which he feels is too great for both his spare time and his advancing years. It is probable, though, that the author will have the book checked some time by one of his assistant engineers, in case that he has to make another design for an arch. Meanwhile he is satisfied to assume that all of the mathematical work is correct, because of Prof. Howe's established reputation as both a mathematician and an engineer. Prof. Howe has tabulated the results of his computations in a very convenient form, so that his formulæ can readily be applied in designing, especially for preliminary designs and estimates. In spite of its discouragingly mathematical appearance, Prof. Howe's book promises to prove of great practical value to designers in structural steel; and its author is certainly to be commended for the immense effort he has put forth in accomplishing for the engineering profession such a laborious piece of work.

Prof. Howe finds the relative weights of metal in a 416′ arch with a 67′ rise, for Cases Nos. 1, 3, and 4, to be as follows:

Case No. 1, no hinges...................	1.00
Case No. 3, two hinges................	1.21
Case No. 4, three hinges..............	1.30

The author is of the opinion that, if he were to make three such designs for comparison, there would not be such great differences in the weights, because constructive reasons will cause the designer to use only a few different sectional areas in the chords of an arch; while Prof. Howe's students, who, as he states, made the calculations from which the tabulated ratios were determined, probably proportioned the section of each panel length of each chord for the greatest stress to which it could be subjected. This would be eminently proper in making such a comparison; but the results of the computations would not agree with similar results obtained by a bridge specialist.

It is difficult to make a proper comparison in respect to economy between arched and simple truss bridges, owing to the fact that the piers differ for the two cases; but a fair one can be obtained by assuming that steel braced piers are used to support the deck span.

The author has had occasion lately to design in complete detail for a British Columbia railroad a 260-ft. arch bridge, shown in Fig. 5, having a rise on the centre line of 59 ft., and to compute the exact weight of metal in same. For the sake of comparison, he has since designed according to the same specifications a 260-ft. deck-span, having a truss depth of 35 ft., resting on steel braced towers 36 ft. high. The total weight of metal for the arch design is 2,111 pounds per lineal foot, and that for the truss design, including the towers, is 2,542 pounds per lineal foot, showing a saving of about twenty per cent in favor of the arch.

As for the relative rigidities of these two structures, there is very little doubt that a comparison of the finished bridges under load would result in favor of the simple span.

In making the preliminary study for the arch bridge herein

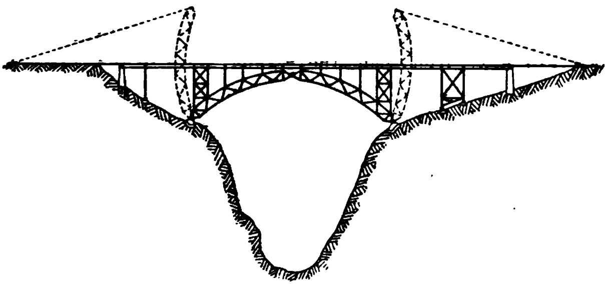


FIG. 5.

referred to, there was prepared a comparative design for a three-hinged arch, in which each half of each arch consists of a lenticular truss as shown in Fig. 6.

Contrary to the author's surmise, this design did not prove

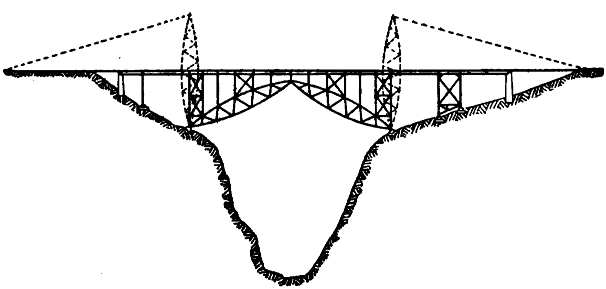


FIG. 6.

to be any more economical than that with the circular arch, the total weights of metal in the two structures being almost exactly the same. The circular arch was, therefore, adopted on account of its superior appearance.

Concerning the relations between the principal dimensions for arch bridges of various types but little can be said, for the reason that but little is known, because of the scarcity of such bridges in this country. In most cases the length of span and the rise are determined by the existing conditions at the crossing. For any given span, the greater the rise the less the effect of uniform load stresses, but the greater the effect of partial load stresses, and *vice versa*. Again, for any given span and rise, the arch depth does not affect the uniform load stresses materially, while it does so affect the partial load stresses; and as the latter are inferior in importance to the former, it results that the depth of an arch for economy of material will be very much less than the best depth for an ordinary truss of the same span. The arch depth, too, will depend upon whether the arch has fixed ends and continuous crown, hinged ends and continuous crown, or hinged ends and hinged crown. For the first type, a varying depth increasing from the centre to the ends is economic; for the second, a varying depth increasing from ends to centre is best; while for the third, a constant depth from end to end seems preferable. Again, the arch depth will depend considerably upon the style of web, i.e., whether it be plate, open-riveted, or pin-connected. The best depth or depths to adopt for any case should be given a special study, in making which Chapter VI of Prof. Howe's book will be found of great assistance.

In respect to the style of curve to adopt, whether circular, parabolic, or elliptical, the author's preference would generally be for the circular on account of its simplicity, although the parabolic might theoretically give better results.

In the author's opinion, a plate-girder arch should be made without hinges, an open-webbed riveted arch either with or without hinges, and a pin-connected arch with hinges. In the latter case, it is only the web members that should be pin-connected, for the chord members should be riveted up and fully spliced from end to end. There should be only a single system of cancellation used in webs of arches, so as to avoid as much as possible ambiguity in the stress distribution. Riveted connections are preferable to pin connections for the diagonals on account of rigidity, but are more expensive for erection.

Hard and fast rules for the minimum spacing of outer arches of bridges for various spans and rises cannot well be given. The narrower the structure within reasonable limits the less the cost, but the less also the rigidity and the lateral resistance to overturning from wind-pressure. In the 260-ft. span herein referred to, the author made the distance between central planes of arches twenty-two feet, which was as small a distance as he dared to adopt, notwithstanding the fact that economy of first cost was an important factor in the design. An approximate rule to work by might be to make the perpendicular distance between outer arches not less than one third of the height from springing point to grade.

In concluding this chapter, the author desires to call attention to the fact that there is still a great deal to be learned about the designing of arches; and to suggest that some professor of civil engineering, who is well posted on bridge designing and who has time to spare, could spend several months to the great advantage of the engineering profession in determining the proper relations of span length, rise, arch depth, width between exterior arches, etc., for the various styles of arch, and in ascertaining the relative economies of the latter.

CHAPTER VII.

TRESTLES AND VIADUCTS.

BUT little need be said in this chapter concerning the designing of trestles and viaducts, as that subject is fully covered in Chapters XIV and XVI. However, as the latter chapters are specifications, and are written in very concise form, it seems advisable to give here certain explanations of the reasons for the rules and directions therein formulated, even at the risk of repetition of a few matters.

The best layout for a trestle or viaduct is the one which will make the cost of the structure a minimum, provided that the specifications used in designing will insure for any layout the requisite strength and rigidity.

As stated in Chapter III, the greatest economy will exist when the cost of the bents and their pedestals is equal to the cost of the longitudinal girders and longitudinal bracing. On this account it is advisable to make the tower spans shorter than the intermediate spans, taking care, however, not to have the former too short for either appearance or proper resistance to traction. In general, tower spans should vary in length from twenty to thirty feet, although for very low structures it may sometimes be advisable to go a few feet below twenty. For the intermediate spans the length generally varies from thirty to sixty feet; but for very low structures with heavy rolling loads the economic length may be found to be less than thirty feet, in which case it will be perfectly legitimate to reduce the span length to suit the economic conditions.

The reason for adopting sixty feet for the superior limit is because trestles and viaducts are nearly always erected without falsework by starting erection at one end of the structure and dropping the members down by means of an overhanging traveller running on top of the erected portion of the work. With tower spans of thirty feet and intermediate spans of sixty feet, the traveller will have to reach out ninety feet to erect a tower, which is about the extreme practicable limit. However, should it be necessary to use more or less falsework, longer spans than sixty feet would probably be economic.

The most economic layout for a highway viaduct with wooden joists is alternate towers and spans that consist entirely of joists, the limiting lengths of span being about twenty feet for the towers and twenty-four feet for intermediates, which latter length is the greatest span used in general practice for 4″ × 16″ wooden joists. It is not legitimate in such a design to rely on the wooden joists of the tower spans to act as a part of the longitudinal tower bracing.

In railroad trestles the longitudinal girders should abut against and rivet into the webs of the columns, the latter being bent just below the longitudinal girders when the legs are battered. The author has lately adopted this detail in some trestle designs for a British Columbia railroad, and has found it to be very satisfactory. For double-track structures, the columns at tops are to be spaced a distance equal to the sum of the perpendicular distance between the longitudinal girders of one track and that between centres of tracks, and the legs may be made vertical up to a limit of about twice the perpendicular distance between axes of opposite columns.

For single-track structures, it is generally best to space the longitudinal girders and tops of columns ten feet centres, although an eight-foot spacing is legitimate. The former spacing gives greater rigidity to the structure, but necessitates the use of deeper timber ties. By using very deep ties a greater girder spacing may be adopted; but this is not necessary, unless very long intermediate spans erected on falsework be employed.

It is not worth while to use a batter for columns less than one and a half inches to the foot, and it is never economical to use one greater than three inches to the foot. The smaller the batter the less the total weight of transverse bracing, but the greater the tension stresses on the columns. As a rule, it is best to keep these tension stresses low or even to make them non-existent; but in high trestles it becomes necessary to permit and provide for them.

It is when trestles are on sharp curves that great batters must be used, in order to provide against the overturning tendency of the combined centrifugal force and wind load. In such cases as these with high trestles it becomes necessary to divide up the transverse bracing of the lower portion of the tower by placing short vertical columns in the middle of the bents, and bracing longitudinally between the vertical columns of alternate adjacent bents.

In very high trestles, especially when located on sharp curves, the combinations of column stresses for live load, dead load, traction load, centrifugal load, and wind load run extremely high, and demand great column sections; consequently in such cases it becomes necessary for the designer to use considerable good judgment so as to reduce the total stress to reasonable limits. For instance, the traction stresses can be cut down to less than one half by riveting the longitudinal girders of an intermediate span to the towers at both ends. This reduces the thrust of train owing to the increased length of structure used for determining the equivalent uniform load, and fixes the tops of the towers so as to make a point of contraflexure at mid-height, thus reducing the lever-arm and therefore the bending moment to one half.

Again, unless the grade be heavy, it is often legitimate to assume that the velocity of train is materially lessened by the sharp curve by the time that the train reaches the high portion of the trestle; and, as the centrifugal load varies as the square of the velocity, the stresses from this load will be greatly reduced by the assumption.

Again, the prevailing high winds and the centrifugal loads may act against each other instead of together, and the combination may be lowered in amount by recognizing this fact.

In short, the designer in such a case can use his judgment to great advantage, and thus save considerable metal that is not really needed, although it might be required if a strict adherence to the specifications were enforced.

The best style of bracing for both the longitudinal and transverse faces of the towers consists of stiff diagonals, each formed of four angles with a single line of lacing, all of said diagonals being riveted to the columns and to each other where they intersect by means of plates, and no horizontal struts being used except at top and bottom of towers, where they are necessary to make the bracing a complete system. The panel-points of the longitudinal bracing should coincide with those of the transverse bracing, although near the top of the tower the panels of the latter may be divided on account of the small distance between columns.

In cheap structures, expense can be saved by making the diagonals of the sway-bracing of adjustable rods, and putting in horizontal struts at the panel points, which struts should always be riveted at their ends to the columns; because pin-connected struts do not stiffen the columns sufficiently to warrant the figuring of the latter as fixed at the panel points.

When adjustable diagonals are adopted, the employment of horizontal struts at top and bottom of towers on all four faces is even more imperative than it is when stiff diagonals are used. The author has seen trestles without such bottom struts, in which the columns have been moved considerably out of place by the rods contracting in cold weather and drawing the column feet together. Six months afterwards the rods elongated and hung in festoons, so were promptly tightened up by the bridge inspectors, thus putting them in good condition to repeat the operation six months later, and so on from year to year till the columns were bowed perceptibly out of line.

In all towers, in each plane of the main panel points of the bracing there should be a horizontal system of diagonal adjustable rods to bring the columns and tower to place and line and to retain them there. The use of these systems of horizontal bracing is a *sine qua non* in scientific designing, for their omission will permit the faces of the tower to get out of plane, and thus the metal in the columns will become overstrained.

Whether it is better to arrange the column feet of towers so as to permit of uninterrupted expansion and contraction or to anchor them down fixedly is a mooted question among engineers. The author prefers the former method for the reason that, if all the feet are anchored so as to prevent all motion, either the pedestals will be sprung laterally or the horizontal struts will bulge or be overstrained when the temperature is at its upper extreme range. In determining the method of sliding, one foot of the four should be made fixed in both directions, two should be fixed in one direction only, and the fourth should be free to slide in both rectangular directions.

Occasionally it is advisable to use hinged ends for a solitary bent; but the author generally prefers to fix the feet and let the column spring laterally under changes of temperature, taking care that it be proportioned properly to resist the stresses due to such springing when the same are combined with the other stresses to which the column is subjected. Fixed ends for columns of solitary bents are much more conducive to rigidity of structure than are hinged ends.

The question of sliding ends for longitudinal girders will be treated in the next chapter, which will deal with elevated railroads, the expansion pockets being the same for such structures as for railroad trestles.

The best sections for columns are either two channels laced or four Z bars with a web plate or lacing. If the columns have to carry transverse loads, they should have solid webs instead of lacing, so as to transmit the shear effectively from top to bottom. For light work, four angles in the form of an I with a single line of lacing will suffice.

All columns when spliced should have their splices located about two feet above the panel points of the column bracing. Failure to so locate them will add materially to the cost of erection. All such splices should be made full, more especially when the tension on the column runs high.

In proportioning anchorages, the pedestal weight should be made not less than twice the greatest net uplift from the column, due account being taken of the buoyant effort of the water in case of a possible submergence of pedestal.

CHAPTER VIII.

ELEVATED RAILROADS.

THE author has lately written for the American Society of Civil Engineers a lengthy paper on this subject. It has been very thoroughly discussed by the engineering profession, and the discussions have been answered in an exhaustive résumé by the author and his assistant engineer, Ira G. Hedrick, Assoc. M. Am. Soc. C. E. The original paper, the discussions, and the résumé have been published in the *Transactions* of the Society for 1897, Vol. XXXVII; and any one who desires to make a special study of the subject of elevated railroads will do well to read all that has been published thereon in the said *Transactions*.

There will, however, be given in this chapter a compendium of the contents of the paper for the use of those who have no time or inclination to wade through the two hundred pages that it occupies.

LIVE LOADS.

The proper live load to assume in designing an elevated railroad is the greatest that can ever come upon it, and is determined by ascertaining the weights of engines and empty cars that are adopted at the outset, then computing how many passengers can be crowded into the latter and assuming that the average weight per passenger is one hundred and forty pounds. The live loads for elevated railroads, unlike those for surface railroads, do not increase from time to time, but remain constant. In fact the late tendency to operate the roads by electricity rather decreases them, for the weight on the axles of a motor car produces smaller bending moments than that on the axles of a locomotive.

After the distribution of the live load on the various axles of the entire train has been determined, it is well to prepare a diagram of equivalent uniform loads and one of total end shears similar to those for the Compromise Standard System of Live Loads for Railway Bridges given in Chapter XIX, in order to facilitate the computing of stresses and bending moments.

Floor.

The style of floor in general use on elevated railroads consists of timber ties with four lines of timber guard-rails, closed floors of buckled plate carrying timber ties in ballast being employed at crossings of important streets and boulevards, so as to prevent dirt and moisture from falling upon people passing beneath. Such a closed floor has been advocated for the entire line, and certainly it would be an improvement upon the open floor; but the increased expense involved is likely to interfere seriously with its adoption for future elevated railroads The ballast over the buckled plate in tight floors is necessary to prevent noise from passing trains, which, unless some effective sound-deadener be adopted, would be simply deafening. There is one important incidental advantage in employing a closed floor, viz., that the elevation of the grade is thereby reduced about three feet. Of course nearly as great a reduction of elevation can be obtained with the open floor by resting the timber ties on the inner bottom flanges of the longitudinal girders; but this style of structure is objectionable for several important reasons, prominent among which are the necessarily large sections of the ties and the difficulty in replacing them.

Economic Span Lengths.

With the ordinary live loads, for structures located on private property the economic span length is about forty feet, while for structures located in the street it varies from forty-seven to fifty-three feet according to the transverse distance between vertical axes of columns, the greater the distance the greater the economic span length. With heavier live loads the economic span lengths would be shorter.

Four-column *versus* Two-column Structures.

In four-track structures located on private property there is but little, if any, difference in the cost whether four columns or two columns per bent be employed; but preference is given to the former on account of rigidity.

Braced Towers *versus* Solitary Columns.

In structures on private property there is quite a gain in both rigidity and economy by adopting braced towers spaced about one hundred and fifty feet centres.

Rails.

The author prefers to adopt for elevated railroads steel rails five inches high weighing not less than eighty pounds to the yard, so as to provide for the excessive wear caused by the constantly passing trains.

Treated *versus* Untreated Timber.

Extended investigations have proved to the author's satisfaction that it pays well to preserve the track timber, and that, up to the present time, by far the best preservative process is vulcanizing.

Pedestal-caps.

The most satisfactory and economical pedestal-caps are of concrete covered with at least six inches of first-class granitoid. These have all the advantages of cut-stone blocks, and are generally cheaper. The latter, however, can be used if there be anything to be gained thereby, provided that the quality of the stone be first-class in every particular.

Anchorages.

The author prefers to anchor columns to pedestals by means of anchor-bolts, either two or four per column, according to whether there is bending on the latter in one or in two directions, extending well down into the concrete and held therein by a cast-iron spider, and extending well up outside of the column, to which they connect by means of long enclosing plates and heavy washers. The boxed spaces at the column feet should always be filled with concrete to prevent the collection of dirt and moisture.

Plate Girders *versus* Open-webbed, Riveted Girders.

As far as economy goes, there is no material difference between plate-girder work and open-webbed, riveted work; but the former is more satisfactory in most particulars, the only real advantage of the latter being that it is more sightly. On this account it is preferable for structures occupying the streets, while plate-girder work is more advantageous for structures located on private property.

Crimping of Web-stiffening Angles.

Investigation has shown that it is economical to crimp intermediate stiffening angles and to use fillers beneath all end stiffeners.

Sections for Columns.

The best section for columns located in the street is composed of two channels with their flanges turned inward and an I beam riveted between the channels, the flanges of the latter being held in place by interior stay-plates spaced about three feet centres. The main object in turning the flanges inward is to enable the column better to resist impact from heavily loaded vehicles.

The most satisfactory section for columns located on private property consists of four Z bars and a web plate.

EXPANSION JOINT.

The author's ideal expansion pocket is described very fully by both text and drawings in his paper on Elevated Railroads, to which the reader is referred.

PROPER DISTANCE BETWEEN EXPANSION POINTS.

With columns fixed at both top and bottom, as the author recommends, the proper distance between expansion points is about one hundred and fifty feet.

SUPERELEVATION ON CURVES.

Superelevation of the outer rail can be obtained by varying the heights of the stringers, by putting a wooden shim on the outer stringer, by using bevelled ties, or by spiking a shim to each tie. The last two methods are generally preferable, but the second one can occasionally be used to advantage, while the first one would give unnecessary trouble in the shops. It will generally suffice to employ only three bevels for ties, viz., one, two, and three inches in five feet. Such bevels will not, it is true, afford the theoretical superelevation required for the maximum speed on sharp curves; but it must be remembered that it is difficult to maintain high speed on sharp curves, hence the compromise between theory and practice.

FAULTS IN EXISTING ELEVATED RAILROADS.

In concluding his before-mentioned paper, the author made a list of the principal faulty details in existing elevated railroads, thereby provoking much animated discussion; and, as the subject is one of great importance to the designer of future similar structures, that portion of the paper which includes this list will be reproduced here *verbatim*.

I. INSUFFICIENCY OF RIVETS FOR CONNECTING DIAGONALS TO CHORDS OF OPEN-WEBBED, RIVETED GIRDERS.

This defect is more noticeable in old structures than in later ones, especially as the tendency nowadays is very properly to substitute plate-girder for open-webbed construction. In many of the older elevated roads there is no connecting plate between the diagonal and the chord, but one leg of each of the angles in the diagonal is riveted directly to the vertical legs of the chord angles. This detail involves the use of either two or four rivets to the connection, which is evidently very bad designing, as there should be more rivets used, even if the diagonal stresses do not call for more on purely theoretical considerations. Where the theoretical number of rivets is very small, additional rivets should be used for two reasons, viz.: first, one or more of the rivets are liable to be loose, and, second, there is nearly always a torsional moment on each group of rivets, owing to eccentric connection.

II. FAILURE TO INTERSECT DIAGONALS AND CHORDS OF OPEN-WEBBED GIRDERS ON GRAVITY LINES.

It is very seldom indeed that the designer even attempts to intersect at a single point all of the gravity lines of members assembling at an apex. The failure to do so involves large secondary stresses, especially in the heavier members. By using connecting plates, it is always practicable to obtain a proper intersection; and it is always better to do this than to try to compensate for the eccentricity by the use of extra metal for the main members.

III. FAILURE TO CONNECT WEB ANGLES TO CHORDS BY BOTH LEGS.

Some standard bridge specifications stipulate that in case only one leg of an angle be connected, that leg only shall be counted as acting, although this stipulation is generally ignored by the designer working under such specifications.

It is seldom, indeed, that both legs are connected. In order to settle the question of the necessity for this requirement, the author has had made, in connection with his Northwestern Elevated work, a series of tests to destruction of full-sized members of open-webbed girders, attached in the testing machine as nearly as practicable in the same way as they would be attached in the structure. It was intended to settle by these tests the following points: first, effect of connecting by one leg only; second, effect of eccentric connection; and, third, the ultimate strength of star struts with fixed ends, each of these struts being formed of two angles. As these tests are not yet finished, their results cannot be given here. The principal deduction to be made from the tests thus far completed is that an equal-legged angle riveted by one leg only will develop about 75% of the strength of the entire net section, while a 6″ × 3½″ angle riveted through the longer leg will develop about 90%. It is therefore more economical for short diagonals to use unequal-legged angles connected by the longer leg than to employ supplementary angles to try to develop the full strength of the piece. In fact, the experiments made up to date indicate that these supplementary angles will not strengthen the diagonal essentially. However, further experiments may show the contrary.

IV. FAILURE TO PROPORTION TOP CHORDS OF OPEN-WEBBED, LONGITUDINAL GIRDERS TO RESIST BENDING FROM WHEEL LOADS IN ADDITION TO THEIR DIRECT COMPRESSIVE STRESSES.

This neglect is common enough in the older structures, and the fault is a serious one, although the stiffness of the track rails and that of the ties tend to distribute the load and thus reduce the bending.

V. INSUFFICIENT BRACING ON CURVES.

Too often in the older structures the curved portions of the line are no better braced than are the straight portions. A substantial system of lateral bracing on curves extending over the entire width of the structure and carried well into the tops of the columns adds greatly to the rigidity of the structure, and, consequently, to the life of the metal-work.

VI. INSUFFICIENT BRACING BETWEEN ADJACENT LONGITUDINAL GIRDERS.

The function of the bracing between longitudinal girders is an important one, for it is the first part of the metal-work to resist the sway of trains. Not only should the top flanges of adjacent girders be connected by rigid lateral bracing, but the bottom flanges should be stayed by occasional cross-bracing frames, one of the latter being invariably used at each expansion end of each track.

VII. PIN-CONNECTED, PONY-TRUSS SPANS AND PLATE GIRDERS WITH UNSTIFFENED TOP FLANGES.

These defective constructions are noticeable in some of the older lines, but, fortunately, not often in the newer.

What the ultimate resistance of the pony-truss structure is no man can tell without testing it to destruction; but, in the opinion of most engineers, it is much less than it is assumed to be by those designing pony-truss bridges.

VIII. EXCESS OF EXPANSION JOINTS.

Too many expansion joints in an elevated railroad are nearly as bad as too few. In the former case the metal is overstrained by the vibration induced by the lack of rigidity, while in the latter case it is overstrained by extreme variations of temperature. There are elevated roads in existence with expansion joints at every other bent, and there is at least one with them at every bent. For long spans there should be expansion provided at every third bent, and for short spans at every fourth bent.

IX. RESTING LONGITUDINAL GIRDERS ON TOP OF CROSS-GIRDERS WITHOUT RIVETING THEM EFFECTIVELY THERETO.

This is by no means an uncommon detail, especially in the older structures. It is conducive to vibration, and its only advantages are ease of erection and a cheapening of the work by avoiding field-riveting.

X. CROSS-GIRDERS SUBJECTED TO HORIZONTAL BENDING BY THRUST OF TRAINS.

The resistance that can be offered by a cross-girder to horizontal bending is very small; nevertheless, cross-girders are rarely protected from the bending effects of thrusts of trains. What saves these cross-girders from failure is the fact that continuity of the track tends to distribute the thrust over a number of bents. Nevertheless, it is not legitimate to depend on this fact, for, especially on sharp curves, the tendency is to carry the thrust into the ground as directly as possible. In the author's opinion, the only proper way to provide for this thrust is to assume that 20% of the greatest live load between two adjacent expansion points will act as a horizontal thrust upon the columns between these two expansion points; and all parts of the metal-work should be proportioned to resist this thrust properly.

By running a strut from the top of each post diagonally to the longitudinal girder at a panel point of its sway-bracing, the horizontal thrust is carried directly to the post, and a horizontal bending moment on the cross-girder is thus prevented. Such construction should invariably be used where the conditions require it.

XI. CUTTING OFF COLUMNS BELOW THE BOTTOM OF CROSS-GIRDERS AND RESTING THE LATTER THEREON.

This style of construction, which until lately was almost universal, is extremely faulty in that there is no rigidity in the connection, and the column is thus made more or less free-ended at the top.

It has been said that no harm is done to the column by making it free-ended, as it can then spring better when the thrust is applied. Unfortunately this reasoning is fallacious, because the few unlucky rivets which connect the bottom of the cross-girder to the top of the column tend to produce a fixed end, and are, in consequence, racked excessively by the thrust of the train. In all cases the column should extend to the top of the cross-girder, and should be riveted to it in the most effective manner practicable.

XII. PALTRY BRACKETS CONNECTING CROSS-GIRDERS TO COLUMNS.

Brackets are often seen composed of a couple of little angles attached at their ends by two or three rivets. Such brackets are merely an aggravation, and are sure to work loose sooner or later. Although it is impracticable to compute the stresses in this detail, good judgment will dictate the use of solid-webbed brackets riveted rigidly to both cross-girder and column so as to stiffen the latter and check the transverse vibration from passing trains.

XIII. PROPORTIONING COLUMNS FOR DIRECT LIVE AND DEAD LOADS AND IGNORING THE EFFECTS OF BENDING CAUSED BY THRUST OF TRAINS AND LATERAL VIBRATION.

The practical effects of this fault can be seen to best advantage by standing on one of the high platforms of one of the elevated railroads of New York City. The vibration, by no means small, from an approaching train can be felt when it is yet at a great distance. Some may claim that this vibration is not injurious; but they are certainly wrong, for what does it matter, so far as the stress in the column is concerned, whether the deflection be caused by vibration or by a statically applied transverse load, so long as the amount of the deflection is the same in both cases? It takes metal, and considerable of it, to make columns strong enough to resist bending properly; and a sufficient amount should be used to attain this end.

XIV. OMISSION OF DIAPHRAGM WEBS IN COLUMNS SUBJECTED TO BENDING.

If the diaphragm web be omitted in such a column, reliance must be placed on the lacing to carry the horizontal thrust from top to bottom. But even if the lacing figure strong enough to carry it, which is unusual, it is wrong to assume it so, for the reason that one loose rivet connecting the lacing-bars will prevent the whole system from acting, as will also a lacing-bar that is bent out of line. Decidedly every column that acts as a beam also should have solid webs at right angles to each other.

XV. INEFFECTIVE ANCHORAGES.

On account of both rigidity and strength, every column ought to be anchored so firmly to the pedestal that failure by overturning or rupture would not occur in the neighborhood of the foot, if the bent were tested to destruction. The flimsiness of the ordinary column-foot connection is beyond description.

XVI. COLUMN FEET SURROUNDED BY AND FILLED WITH DIRT AND MOISTURE.

The condition of the average column-foot is simply deplorable. This is caused by failing to raise it so high above the street as to prevent dirt from piling around it, and by omitting to fill its boxed spaces with concrete. When rusting at a column-foot is once well started, it is almost impossible to stop it from eating up the metal rapidly.

XVII. INSUFFICIENT BASES FOR PEDESTALS.

False ideas of economy on the part of projectors and indifference on the part of some unscrupulous contractors occasionally cause the use of pedestal bases altogether too small for the loads that come upon them, especially where the bearing capacity of the soil is low. The result is sunken pedestals and cracked metal-work. In figuring the pressure on the base of the pedestals, it is not sufficient to recognize only the direct live and dead loads, but it is necessary also to compute the additional unequal intensities of loading caused by both longitudinal and transverse thrusts.

Concerning the question of the extent to which the faults just outlined exist in the older elevated railroads of this country, the author would refer the reader to the résumé of discussions on his paper, and to the report of Mr. Hedrick which it contains.

About the most important object to attain in constructing an elevated railroad is to have a perfectly smooth and durable track; and no trouble or expense should be spared to secure it. For this reason the top flanges of the longitudinal girders, if the limiting heights of grade and clearance line permit, should be several inches higher than those of the cross-girders, the ties should all be planed to exact dimensions tie-plates should be used over all ties, and the system of bolting of flooring to structure should be the most effective possible. The longitudinal girders should not be made continuous, or even semi-continuous over the cross-girders, but, when blocking up is necessary, short buckled plates should be placed over the latter so as to provide a continuous surface for the ties. Hook bolts with cold-pressed threads should be used for attaching the timber to the metal-work through each alternate tie, the other ties being bolted to the inner guard-rails.

The ties should be spaced with openings not greater than six inches, their section for a five-foot stringer being 6″ × 8″ laid on flat; but where cross-overs are employed, the depth should be properly increased to withstand the bending moment due to the greatest load from the wheels.

The least allowable overhead clearance for most cities is fourteen feet; but there are sometimes special crossings requiring a greater height. The width of right of way beyond the centre line of the outer track should not be less than seven feet. The proper depths of longitudinal girders are to be determined very carefully. For the sake of appearance it is generally not well to use more than one depth, but such an arrangement cannot always obtain. The general depth should, if possible, be the economic one for the average span length. For plate-girder spans it is about one twelfth of the length, while for open-webbed, riveted spans it is much greater—so much greater, in fact, that for deck-spans the economic depth cannot be adopted, because of the raising of the grade which would be caused thereby.

Before the designing of the metal-work for an elevated railroad is started there are certain important matters which should be fully determined, viz., the dimensions and weights of rolling stock, sizes and number of trains, method of traction and the proper track to suit same, the locations of all stations and their leading dimensions, the storage capacity for the terminals, the capacity of the repair-shops, and the method of operating the road. Unless all these questions be settled conclusively at the outset and before the designing is begun, trouble is sure to ensue because of changes that will have to be made from time to time during the course of construction.

In designing elevated railroads according to the specifications given in this treatise, it must not be forgotten that the entire line is to be proportioned by the specifications for railroad bridges (Chapter XIV), while the stations are to be proportioned by the specifications for highway bridges (Chapter XVI).

CHAPTER IX.

MOVABLE BRIDGES IN GENERAL.

MOVABLE bridges may be divided into the following eight types:

1. Ordinary, rotating draws.
2. Double, rotating, cantilever draws.
3. Pull-back draws.
4. Counterweighted, bascule bridges.
5. Rolling, bascule bridges.
6. Jack-knife or folding bridges.
7. Lift-bridges.
8. Floating bridges.

The ordinary rotating draws will be treated at length in the next chapter.

Very few double, rotating, cantilever draws have yet been built; in fact the author knows of but one, viz., that over the canal at Cleveland, Ohio. A number of years ago the author had occasion to figure on a large structure of this kind, but it was never built.

The principal advantages of this type of structure are a wide waterway and the retreating of the span without serious injury when struck by a vessel before it is fully opened; while its disadvantages are excessive first cost and the almost double cost of operating two independent spans; although, when electricity is used as a motive power, both spans can be operated by one man by means of a submerged cable.

This class of bridge consists of two draw-spans, differing but little from the ordinary rotating draw, each resting upon a pivot-pier and meeting at mid-channel, where they are locked together so as to make the adjoining ends deflect equally and simultaneously. The other end of each draw is locked to the masonry of the outer rest-pier, which acts as an anchorage. It is not necessary to make the shore arms of the same length as the channel arms ; but if there be a difference, there must be compensating weight so as to balance each span over the centre of the pivot-pier, and there must be a vertical, close surface provided at the end of each short arm so as to equalize as well as possible the moments of the wind-pressure on the two arms. This class of bridge is probably not very rigid, but it can be made quite satisfactory and effective.

The pull-back draw is also a very unusual type, and will always be so, for the reason that the first cost is great and its operation is expensive. This type may be divided into two classes: first, structures with one span over the entire opening, and, second, structures with two spans over the entire opening, meeting at mid-channel, as in the case of the double, rotating, cantilever draw. The first class requires a truss-bridge nearly, if not quite, twice as long as the width of channel between pier centres, the bottom chords thereof running on two groups of rollers that travel just half as fast as the bridge when the span is moved longitudinally. Although the shore arm may be made shorter than the channel arm, still its weight must be such that its moment will be somewhat greater than the tipping moment of the weight of the channel arm just as it leaves the farther pier. A disappearing platform will be required so as to leave space on the approach for the shore arm to move back, or else the whole bridge will have to be rotated slightly about a horizontal axis so that it can roll up onto the approach. Either method is very clumsy, and the operation of the bridge consequently must be slow.

The double pull-back draw is similar to the single pull-back draw just described, except that the far end of each span has to be anchored down to a mass of masonry when the bridge is closed and ready for traffic, and the ends meeting at mid-channel must be locked together as in the case of the double, rotating, cantilever draw.

The author had occasion several years ago to design a double, pull-back drawbridge; and although he certainly evolved a structure that would work, he was far from satisfied with the design, so recommended another type of bridge for the crossing.

There is described in the *Engineering Record* of July 31, 1897, a double pull-back draw, which has just been completed over the River Dee at Queensferry, Scotland. It provides a clear opening of one hundred and twenty feet, and cost about $70,000.

Bascule bridges are those in which a shallow deck is raised from a horizontal position to a vertical or inclined one so as to let vessels pass. They may have either one or two leaves whose weight may be counterbalanced in various ways. When two leaves are used, they may be made to meet at mid-channel and form an arch, may rest on a central pier, may hang from a tower or from an overhead span, or may have hanging, hinged bents to rest on a submerged pier at the elevation of the bed of the channel.

For spans requiring leaves not longer than seventy-five feet the bascule type of bridge is very satisfactory; but beyond that limit the first cost of the structure begins to get too high as compared with another type of equally satisfactory structure, viz., the lift-bridge. Some bascule bridges, notably the Tower Bridge of London, England, have an overhead span to be used in connection with elevators by pedestrians when the lower deck is opened for the passage of vessels. The leaves of the Tower Bridge, which are each 113 feet long, do not raise quite to a vertical position, requiring one and a half minutes to open and as much more to close under favorable conditions of wind and weather, and sometimes twice as long when the conditions are unfavorable. The author has been told by an English gentleman resident in America, who made lately an investigation concerning the Tower Bridge, that the London people complain bitterly about the long time it takes to operate the structure. A lift-bridge similar to the Halsted Street Lift-bridge of Chicago, Ill., could have been built instead, which would raise to full height in from thirty to forty-five seconds and lower again in the same time. A full description of the Tower Bridge is given in the *Proceedings* of the Institution of Civil Engineers, Vol. CXXVII.

A good example of the rolling bascule bridge is the Van Buren Street Bridge at Chicago, Ill., which structure is described in *Engineering News* of Feb. 21, 1895. It consists of two leaves, each about seventy feet long, ending in a cylindrical surface that rolls on a plane provided with teeth which gear into the roller to prevent slipping. When the bridge is closed the short end of each arm is anchored down to the masonry so as to permit of its acting as a cantilever.

Close alongside of this structure is the Metropolitan Elevated Railroad Company's four-track bridge, which is also of the rolling bascule type. This is divided into two similar double-track bridges placed close together and operated separately, so that, in case of accident to one bridge, the railroad traffic may be diverted to the other, while the injured span is raised out of the way of the river traffic.

It is seldom advisable to use a centre pier to rest the leaves of the bascule upon, on account of the obstruction which it would offer to navigation.

A submerged pier to receive the ends of the posts of hanging hinged bents has never been used, nor is it at all likely that it ever will be, owing to the difficulties that would be encountered in operation, such as those from ice, drifting sand, changing currents, etc., all of which would tend to prevent the column-feet from taking proper bearing on the pier.

Suspending the ends of the leaves from an overhead span, or tying them back to the tops of towers, is a perfectly feasible method, but is expensive and without advantage.

There is a bascule bridge in Chicago which is counterweighted by four masses of cast iron in carriages that run upon curved surfaces on the approaches, the curves being so figured that the varying load at the channel end of the leaf is at all times balanced by the varying tension on the cables which hold the counterweights.

There is a similar structure on Michigan Avenue in Buffalo,

N. Y., which is described in the *Engineering Record* of Aug. 21, 1897.

Several years ago the author figured on a bridge of this type, but abandoned the design because he deemed it inferior to several others which he prepared for the same crossing.

An excellent type of bascule bridge is that of the Sixteenth Street Bridge over the Menominee Canal at Milwaukee, Wis. It is described in *Engineering News* of March 7, 1895. The peculiar feature of this design is that, during motion, the centre of gravity of the mass travels in a horizontal plane, thus reducing to zero the lifting effort for the machinery.

A temporary bascule of peculiar detail was used for several years at the crossing of the Harlem River on the New York Central and Hudson River Railroad. It is described in the *Railroad Gazette* of June 10, 1892. The characteristic feature of this structure, which by the way is a rather clumsy contrivance, is the picking up and dropping of small counterweights while lowering or raising the span.

There is a serious objection to all large bascule bridges, viz., the great surface opposed to the wind by the leaf or leaves when the bridge is being opened or closed. To overcome this pressure powerful machinery has to be used; and it is by no means improbable that even such machinery will be stalled when a high wind prevails.

The jack-knife or folding bridge is a type of structure which is not at all likely to become common. There have been only two or three of them built thus far, and they have been often out of order; moreover, considering the size and weight of bridge, the machinery used is powerful and expensive. The load on the machinery while either opening or closing the bridge is far from uniform, and the structure at times almost seems to groan from the hard labor. The characteristic feature of the jack-knife bridge is the folding of the two bascule leaves at mid-length of same when the bridge is opened. The loose-jointedness involved by this detail is by no means conducive to rigidity; nevertheless these structures are stiffer than one would suppose from an examination of the drawings. The Canal Street Bridge, Chicago, is of this type; and its design is illustrated in *Engineering News* of December 14, 1893.

Lift-bridges on a small scale have been used for many years for crossings of canals, lifting only high enough to let the canal-boats pass beneath. They have proved to be quite satisfactory and fairly economical in both first cost and operation, the method of the latter being usually man-power.

No large structure of this type was ever built until 1893, when the author designed for the city of Chicago the South Halsted Street Lift-bridge. This structure has been described in the principal engineering papers of America and Europe, and the author's description, written for the American Society of Civil Engineers, may be found in the *Transactions* of that Society for January 1895, from which the following description is taken :

The bridge consists of a single, Pratt truss, through-span of 130 ft., in seven equal panels, and having a truss depth of 23 ft. between centres of chord pins, so supported and constructed as to permit of being lifted vertically to a height of 155 ft. clear above mean low water. At its lowest position the clearance is about 15 ft., which is sufficient for the passage of tugs when their smokestacks are lowered. The span differs from ordinary bridges only in having provisions for attaching the sustaining and hoisting cables, guide-rollers, etc , and in the inclination of the end posts, which are battered slightly, so as to bring their upper ends at the proper distance from the tower columns, and their lower ends in the required position on the piers.

At each side of the river is a strong, thoroughly braced, steel tower, about 217 ft. high from the water to the top of the housing, exclusive of the flag-poles, carrying at its top four built-up steel and cast-iron sheaves, 12 ft. in diameter, which turn on 12-in. axles. Over these sheaves pass the 1½-in. steel-wire ropes (32 in all) which sustain the span. These ropes are double, i.e., two of them are brought together where the span is suspended, and the ends are fastened by clamps, while, where they attach to the counterweights, they form a loop, which passes around a 15-in. wheel or pulley that acts as an equalizer in case the two adjacent ropes tend to stretch unequally.

The counterweights, which are intended to just balance the weight of the span, consist of a number of horizontal cast-iron blocks about 10 × 12 in. in section, and 8 ft. 7 in. long, strung on adjustable wrought-iron rods that are attached to the ends of rockers, at the middle of each of which is inserted the 15-in. equalizing wheel or pulley previously mentioned.

The counterweights run up and down in guide-frames built of 3-in. angles.

The weight of the cables is counterbalanced by that of wrought-iron chains, one end of each chain being attached to the span and the other end to the counterweights, so that, whatever may be the elevation of the span, there will always be the same combined weight of sustaining cables and chain on one side of each main sheave as there is upon its other side.

Between the tops of the opposite towers pass two shallow girders thoroughly sway-braced to each other, and riveted rigidly to said towers. The main function of these girders is to hold the tops of the towers in correct position; but incidentally they serve to support the idlers of the operating ropes and to afford a footwalk from tower to tower for the use of the bridge-tender. Adjustable pedestals under the rear legs of each tower provide for unequal settlement of the piers which support the tower columns. Each of these pedestals has an octagonal forged steel shaft, expanding into a sphere at one end, and into a cylinder with screw-threads at the other. The ball end works in a spherical socket on a pedestal, and the screw end works in a female screw in a casting which is very firmly attached to the bottom of the tower-leg. By turning the octagonal shaft, it is evident that the rear column will be lengthened or shortened. The turning is accomplished by means of a special bar of great strength, which fits closely to the octagon at one end, and to the other end of which can be connected a block and tackle if necessary. These screw adjustments were useful in erecting the structure, but it is quite likely that they will never again be needed. But in case there is ever any tower adjustment required, it will be found that the extra money spent on them will have been well expended.

Each tower consists of two vertical legs, against which the roller-guides on the trusses bear, and two inclined rear legs. These legs are thoroughly braced together on all four faces of the tower; and at each tier thereof there is a system of horizontal sway-bracing, which will prevent most effectively every tendency to distort the tower by torsion.

At the tops of the towers there are four hydraulic buffers that are capable of bringing the span to rest, without jar, from its greatest velocity, which was assumed to be 4 ft. per second; and there are four more of these buffers attached beneath the span, one at each corner, to serve the same purpose.

The span, with all that it carries, weighs about 290 tons, and the counterweights weigh, as nearly as may be, the same. As the cables and their counterbalancing chains weigh fully 20 tons, the total weight of the moving mass is almost exactly 600 tons.

Should the span and the counterweights become out of balance on account of a greater or less amount of moisture, snow, dirt, etc., in and on the pavement and sidewalks, it can be adjusted by letting water into and out of ballast-tanks located beneath the floor; and, should this adjustment be insufficient, provision is made for adding small weights to the counterweights, or for placing such weights on the span.

As the counterweights thus balance the weight of the span, all the work which the machinery has to do is to overcome the friction, bend the wire ropes, and raise or lower any small unbalanced load that there may be. It has been designed, however, to lift a considerable load of passengers in case of necessity, although the structure is not intended for this purpose, and should never be so used to any great extent.

The span is steadied while in motion by rollers at the tops and bottoms of the trusses. There are both transverse and longitudinal rollers, the former not touching the columns, unless there is sufficient wind-pressure to bring them to a bearing. The longitudinal rollers, though, are attached to springs, which press them against the columns at all times, and take up the expansion and contraction of the trusses. With the rollers removed, the bridge swings free of the columns; and, since the attachments are purposely made weak, the result of a vessel's striking the bridge with its hull will be to tear them away and swing the span to one side. Should the rigging of the vessel, however, strike the span, the effect will be simply to break off the masts without injury to the bridge. This latter accident has happened once already, the result being exactly what the author had predicted. There is a special apparatus, consisting of a heavy square timber set on edge, trimmed on the rear to fit into a steel channel which rivets to the cantilever-brackets of the sidewalk, and faced with a 6 × 6-in. heavy angle-iron, to act as a cutting edge. This detail is a very effective one for destroying the masts and rigging of colliding vessels.

The bridge is designed to carry a double-track street railway, vehicles, and foot-passengers. It has a clear roadway of 34 ft. between the counterweight guides in the towers, the narrowest part of the structure, and two cantilevered sidewalks, each 7 ft. in the clear, the distance between central planes of trusses being 40 ft., and the extreme width of suspended span 57 ft., except at the end panels, where it is increased gradually to 63 ft. The roadway is covered with a wooden block pavement 34 ft. wide between guard-rails resting on a 4-in. pine floor, that in turn is supported by wooden shims which are bolted to 15-in. I-beam stringers, spaced about 3 ft. 3 in. from centre to centre. These stringers rivet up to the webs of the floor-beams, and beneath them run diagonal angles, which rivet to the bottom flange of each stringer, and thus form a very efficient lower lateral system. The sidewalks are covered with 2-in. pine planks, resting on 3 × 12-in. pine joists spaced about 2 ft. from centre to centre.

The span is suspended at each of the four upper corners of the trusses by eight steel cables, which take hold of a pin by means of cast-steel clamps. This pin passes through two hanger-plates which project above the truss, and are riveted very effectively to the end post by means of the portal plate-girder strut on the inside and a special, short, cantilever girder on the outside.

Each portal-girder carries near each end an iron-bound oak block to take up the blow from the hydraulic buffer, which hangs from the overhead girder between towers. Similar oak blocks are let into and project from the copings of the main piers to take up the blow from the hydraulic buffers that are attached to the span.

The ballast-tanks before alluded to, of which there are four in all, are built of steel plates properly stiffened, and have a capacity of about 19,000 pounds, which is probably more than enough to set the bridge in motion, if it were all an unbalanced load. These tanks serve a double purpose, the first being simply to balance the bridge when it gets out of adjustment because of the varying load of moisture, etc., on the span, and the second being to provide a quick and efficient means of raising and lowering the span in case of a total breakdown of the machinery. If, for instance,—which is highly improbable,—the operating ropes were broken and had to be detached from their drums, by emptying all of the water out of the tanks the span could be made to rise. It could be lowered again by filling them from a reservoir which is placed on top of one of the towers and kept filled with water at all times by means of a pump in the machinery-house. The water in all of these tanks can be kept from freezing, or the ice therein can be thawed at any time, by turning on steam from the machinery-room into the coils of pipe which they contain.

The operating machinery is located in a room 37 × 53 ft., the opposite sides being parallel, but the adjacent sides being oblique to each other, the obliquity amounting to about 12 degrees. The placing of this machinery beneath the street was really forced upon the author, who had originally contemplated using electrical machinery and putting it in a house in one of the towers.

The arrangement of the operating machinery is as follows: Two 70-H.P. steam-engines communicate power to an 8-in.

horizontal shaft carrying two 6-ft. spiral-grooved, cast-iron drums, around which the $\frac{7}{8}$-in. steel-wire operating cables pass. As one of the lifting-ropes passes off the drum, the corresponding lowering-rope takes its place, and *vice versa*, the extreme horizontal travel being a little less than 12 in. Thus by turning the drums in one direction the span is raised, and by turning them in the other direction the counterweights are raised, and the span consequently is lowered. When the span is at its lowest position, the full power of one engine can be turned on to pull up on the counterweights, thus throwing some dead load on the pedestals of the span, after which the drums can be locked. Before the bridge was completed the writer considered that this would be necessary, in order to check vibration from rapidly passing vehicles; but such has not proved to be the case, for the span is very rigid, and the amount of the vibration is not worth mentioning. It is possible, though, that in some other lift-bridges, where the ratio of live load to dead load is greater, this feature of operation cannot be ignored.

The engines are provided with friction-brakes that are always in action, except when the throttle is opened to move the span; consequently no unexpected movement of the span is possible.

The raising-ropes, after leaving the drums, pass out of the machinery-house to and beneath some 5-ft. idlers under the towers, thence up to the top of the north tower, where they pass over some 4-ft. idlers and the main 12-ft. sheaves. Four of them here pass down to the north end of the span, and the other four run across to the other tower over more idlers, then down to the south end of the span.

The lowering-ropes, after leaving the drums in the machinery-room, pass under some idlers below the north tower, and thence up to more idlers at the top of the tower. Four of them here pass down to the counterweights in the north tower, and the other four run across, over intermediate idlers in the overhead bracing, to the main 12-ft. sheaves of the south tower, then downward to the counterweights.

In addition to the previously mentioned method of moving the span by the water-ballast, there is a man-power operating apparatus of simple design in the machinery-house, which, when used alone, can raise and lower the span slowly in case the steam-power gives out, or more rapidly when combined with the water-ballast method.

As the span nears its highest and lowest positions, an automatic cut-off apparatus in the machinery-room shuts off the steam from the cylinders and thus prevents the hydraulic buffers from being overtaxed.

ADVANTAGES OF LIFT-BRIDGES.

The advantages of lift-bridges in comparison with rotating drawbridges are as follows:

1st. A lift-bridge gives one wide channel for vessels instead of the two narrow ones afforded by a centre-pivoted swing-bridge.

2d. There are no land damages in the case of a lift-bridge, as the whole structure is confined to the width of the street. These land damages in the case of some swing-bridges amount to a large percentage of the total cost of structure.

3d. Vessels can lie at the docks close to a lift-bridge, which they cannot do in the case of a swing-bridge; consequently with the former the dock-front can be made available for a much greater length between streets than it can with the latter.

4th. The time of operation for a lift-bridge is about 30% less than that for a corresponding swing-bridge.

The advantages of a lift-bridge in comparison with a bascule or a jack-knife draw, both of these being supposed to be without a centre pier, are as follows:

1st. The lift-bridge can be made of any desired span, while in the case of the others the span is necessarily quite limited in length.

2d. A lift-bridge can be paved, while the others cannot.

3d. The lift-bridge is very much more rigid than any structure composed of two or more partially or wholly independent parts, a feature characteristic of the jack-knife bridge or the bascule without a centre pier.

4th. In a lift-bridge the operating machinery is much more simple; and, in case that it should ever get out of order, the span can be raised or lowered either by unbalancing, or by simple hand mechanism, or by both combined.

If the author were to design another lift-bridge similar to the Halsted Street structure, and if he were given *carte blanche* in the designing, he would make the following improvements:

1. Curve the rear columns and arch the overhead girders at tops of towers, so as to improve the general appearance.

2. Operate by electricity instead of by steam.

3. Place the machinery-house in one of the towers and dispense with the operating-house on the span, letting the

operator stand in a bow-window of the machinery-house so as to command a view of the river in both directions.

4. Omit the water-tanks as an unnecessary precaution, and rely on the great capacity of the electric motors to overcome any temporary unbalanced load.

5. A simpler and less expensive adjustment at feet of rear columns.

6. Cast steel instead of cast iron for all machinery.

7. Catch the balancing chains in buckets placed on top of the span instead of hanging them to the counterweights.

The author has designed a rather peculiar lift-bridge for a crossing of the Missouri River at Kansas City, Mo., at the site of the unfinished Winner Bridge, the piers for which have been completed for over six years. The proposed superstructure will provide for two railway-tracks on each deck, a single-track wagonway outside of each truss below, and a footwalk outside of each wagonway. The perpendicular distance between central planes of trusses is to be thirty-two feet.

The requirements of navigation will be provided for by means of a lifting deck in the second channel span from the Kansas City side, suspended from a through overhead span. This span will be supported on steel columns carried by the existing masonry piers, which will have to be cut down to about the elevation of standard high water, then rebuilt for two or three courses. At one end of the supporting span the vertical end posts are made fast to the bent posts below by means of a pin connection, but at the other end there is to be a nest of friction-rollers between the foot of each vertical end post and the top of the bent column beneath. These bents are to be stayed to the inclined end posts of the adjoining spans.

The lifting deck will consist of four lines of railway plate-girder stringers and four lines of open-webbed highway stringers, with an effective system of horizontal and vertical sway-bracing between the stringers of each pair, besides a very rigid lateral system attached to the lower flanges of all stringers. All of these stringers will rivet up against the webs of the cross-girders, the elevations of the upper surfaces of all longitudinal and cross girders being the same, so as to permit of making the longitudinal girders continuous by means of cover-plates. To permit of the use of similar cover-plates for the bottom flanges of the longitudinal girders, the webs of the cross-girders are to be slotted for their passage, and the weakened web sections are to be strengthened by means of angle-irons.

The cross-girders, which are slightly fish-bellied, are to be riveted at their ends into hangers, each of which is composed of two twelve-inch I beams, the distance between the vertical axes of hangers being forty-one feet. Beyond the hangers will be cantilever brackets for carrying the highway stringers, said brackets being connected at top to the cross-girders by cover-plates and at the bottom by planed ends that will afford effective contact for the meeting flanges.

At the top of each hanger is a detail for connecting to the cables, and beneath the same is placed a hydraulic buffer so arranged that, when the movable deck is at its lowest position, the live load thereon is carried by the hangers through the buffers to certain cantilever brackets, which project from the ends of the cross-girders of the supporting span.

These cantilever brackets and the fish-bellying of floor-beams and stringers are the only peculiar features of the supporting span, with the exception of the vertical end posts and the unusual sizes of all truss members.

While the live load of the movable deck is carried through the hydraulic buffers to the bottom of the supporting span, the dead load passes by means of the wire cables to the top of said span.

The lifting deck will be operated by electrical machinery located in the house at the middle of the top of the through or supporting span. The weight of the lifting deck, which amounts to about 1,850,000 pounds, is counterbalanced by cast-iron weights in groups, each about four feet long, four feet wide, and four feet six inches high, strung on tightly adjusted rods to hold them in position; and is supported by one hundred and twelve steel-wire cables one and a quarter inches in diameter that pass over fifty-six cast-iron sheaves five feet in diameter. These sheaves are connected by transverse three-inch shafts and gearing to the central four and a half inch shaft, which runs the whole length of the through span and connects to the two one hundred horse-power electric motors in the machinery-house. Either motor alone is capable of operating the lift under the most unfavorable conditions. Each sheave supports the two halves of a wire rope about one hundred and sixty-five feet long, the ends being run into sockets. This rope passes around a twelve-inch equalizing-wheel attached to the counterweight suspender, so as to adjust any unequal stretch of the two halves of the rope.

The hydraulic buffers previously described, thirty in number, are used to bring the deck to rest at the lowest position of its travel, and thirty more are employed for the same purpose at its highest position.

In addition to the buffers there will be automatic, electric cut-offs to remove the power before the deck reaches either end of its travel, besides powerful brakes to bring the moving mass to rest quickly whenever the operator may so desire, *and always automatically at the highest and lowest points of travel*, in order to relieve the buffers.

The main sheaves are five feet in diameter and five inches wide, with eight radial arms. They are each cast in one piece and keyed to a seven-inch steel axle, that rests on two pillow-blocks each eight inches long, fitted with bronze bearings.

The pillow-blocks rest on short posts riveted into long transverse girders that rest on the top chords and cantilever out beyond them about five feet at each end. These posts are to be well braced longitudinally. The supporting detail between the transverse girders and the top chords is such as to distribute the load properly over the latter.

There will be at each of the four corners of the moving deck two rollers for transverse motion and two for longitudinal motion, all acting on the faces of the columns that uphold the supporting span. The transverse rollers do not act unless there be sufficient wind-pressure on the deck to move it laterally; but the longitudinal rollers act whenever the deck is moved, as they are backed by springs that press them at all times against the columns.

When the deck is at its lowest position it will be held firmly to the piers, with a proper provision for longitudinal expansion, in such a manner as to relieve entirely the guide-rollers from carrying the wind-pressure, so that they can act only when the deck is raised.

The machinery-house will be about twenty-two feet square and fourteen feet high under the eaves, capped by a dome, and finished in an ornamental style. The floor is to be of I beams supporting a four-inch plank floor.

All main sheaves are to be covered with ornamental housings, and all gears are to be covered with small galvanized-iron hinged housings.

The velocity of the lifting deck will be limited to one foot per second by means of an automatic governor attached to the electrical machinery. The time required to either raise or lower the deck the full height will therefore be about one minute.

To provide for a possible breakdown of the electrical machinery, a man-power apparatus will be employed, consisting of two capstans connected to the main shaft by means of gearing located in the machinery-house, and operated by levers working in horizontal planes.

The moving deck and counterweights will be balanced when the deck is at mid-height. On this account there will be a constant tendency to hold the deck from vertical motion at both ends of its travel, because of the unbalanced weight of the wire cables. In one sense this will be a decided advantage, but it will necessitate extra power to start the mass in motion. Again, the deck will be balanced for ordinary conditions of weather, but it is probable that the weight will be increased by moisture, accumulated dirt, etc. This, if it exist to a moderate extent, will be an advantage, in that it will tend to hold down the deck on the piers ; but, as before, it will require increased power to start motion and to operate. However, the amount of power available will be large enough to meet all conditions of loading and contingencies. Should the deck become lighter than the counterweights by reason of the drying of the timber in the floor and screens, it will be necessary to add to its weight by loading it; but this condition is not likely to exist, for what weight is lost by drying will be fully made up by accumulated dirt in spite of all the precautions that may be taken to keep the floors clean.

Whether this proposed structure will ever be built is problematical, although there is a fair chance of its being finished some day with modifications tending to cheapen the work. It would be a great satisfaction to the author to complete this bridge because of the novel design for the lifting deck.

Floating draws are a type of structure that cannot be recommended except as a temporary expedient. The author had occasion once to design one of them, but the necessity for its use did not develop, so it was not built. The objections to floating draws are as follows:

1. Trouble from rise and fall of water, necessitating constant adjustments.
2. The depression of the draw under the live load and the consequent changing of the grade.
3. Possible disaster from injury by ice or drift.
4. Trouble from leakage.
5. Clumsiness of method of opening and closing the draw.

As there are no advantages to offset these disadvantages, unless it be possibly a small saving in first cost of span, it is not likely that there will be much call for floating draws.

In concluding this chapter, it may be well to summarize somewhat and indicate what kinds of draws should be used at various crossings.

For streams bearing a moderate amount of traffic with crossings located in country districts or in unimportant cities, rotating draws are the cheapest and consequently the most appropriate; but for great traffic and for important cities bascule and lift bridges are the best, the former for spans up to about one hundred and fifty feet, and the latter for longer spans. The choice between the bascule and the lift for all doubtful cases should be determined simply by the question of first cost.

CHAPTER X.

REVOLVING DRAWBRIDGES.

REVOLVING draw-spans are required when bridges across navigable streams are not high enough above the water to provide the proper vertical clearance for passing vessels. Before taking up the discussion of draw-spans, it will be well to consider the relative advantages and disadvantages of high and low bridges for the crossing of such streams as the Mississippi, the Missouri, and the Arkansas rivers.

As a rule, there is very little difference in the first cost of a high and of a low bridge for such a crossing, what little there is being in favor of the latter and seldom amounting to more than ten per cent. Each pier of a low bridge is cheaper than the corresponding pier of a high bridge; but this saving is offset by the cost of the pivot-pier, which is extra. The superstructure of a low bridge may be a trifle lighter than that of the corresponding high bridge, but the more expensive metal-work of the draw-span generally overbalances this. It is in the low, short trestle approaches that the low bridge costs less than the high one.

As these approaches are generally built of timber, they have to be renewed about once in every eight years, and the cost of renewal is a regular fixed charge, which lessens the annual net income from the bridge.

Herein lies the superiority of the low bridge for such crossings. Nor is this its only advantage, for, by its adoption, there is generally avoided a considerable climb at each end of the structure.

On the other hand, the low bridge involves some expense for operation, which is quite an important matter when there is much river traffic, but which is of slight importance when the draw has to be opened only a few times per season, as is the case with bridges over most Western navigable streams.

Everything considered, whenever there is any choice between a high and a low bridge for the crossing of any important Western river, the author favors the low bridge, not so much because of its lower first cost, but on account of the smaller expense for maintenance.

The different kinds of revolving draw-spans recommended are described in detail in Chapters XV and XVII. They may be operated in various ways, for instance by man-power, steam, electricity, gas or gasoline engines, or water. Wherever an unfailing supply of electricity is available, that source of power is the best and cheapest. Steam is appropriate for large, heavy draws where electricity is not available. Gas or gasoline engines are best suited for comparatively small spans in country districts; and water-power can sometimes be employed to advantage where there is a fall of water near the bridge.

It does not pay to use storage batteries for operating draw-bridges. Concerning this question the author feels that he can speak as an authority, for he once made the experiment, and it was a failure. For a while the machinery worked to perfection, but soon the batteries began to leak, and the leakage gradually increased to such an extent that the batteries would not hold their charge for three consecutive days; so the electrical power was given up, and the bridge has since been operated by hand.

Gasoline engines, everything considered, are probably the best source of power for operating the average draw-span. The author has lately designed some small draws to be operated thereby; but the machinery has not yet been installed, so he cannot report concerning how such engines act.

In respect to the power required to operate draw-spans, the author uses an average of the Boller formulæ, viz.,

$$\text{H. P.} = \frac{0.0125\,W\,v}{550},$$

where $W =$ total load on rollers in pounds, and $v =$ velocity on pitch-circle of rack in feet per second.

The author obtained a fine check on the correctness of this formula when testing the draw-span of his Jefferson City highway bridge. This span of 440′ weighs 660,000 pounds, and was opened by four men in four minutes and fifty seconds. The power applied by the men was measured by dynamometers, and from the length of their path and from their pull the horse-power was computed. It proved to be just a little less than unity, so near in fact that it was called unity. The velocity v was, on the average, 0.066 feet per second. Substituting in the formula gives

$$\text{H. P.} = 0.0125 \times 660{,}000 \times 0.066 \div 550 = 0.99.$$

It is possible that, if the experiments were to be made again, a greater divergence from the formula would be found, for the reason that the bridge is liable to work more easily after it has been operated a while.

The computation of stresses in ordinary draw-spans involves more or less ambiguity. The assumptions upon which the calculations are based are the following:

1. The truss-rods in the tower are so light that they cannot transfer any shear past the pivot-pier; consequently, with a live load on one arm only, the said arm acts entirely independently of the other, thus making the draw for this loading consist of two simple spans.

2. For live load on both arms, the reactions are to be found on the assumption that the draw is a continuous girder on four points of support, and by a formula based upon the Theorem of the Three Moments with a constant moment of inertia.

Plate IX gives a diagram from which can be read at a glance the percentage values of the reactions for any balanced load placed anywhere on any span. It is perhaps theoretically not quite perfect, because the values of the reactions depend slightly upon the ratio of distance between the two middle points of support to length of one arm; but any error made by assuming this ratio as constant for all drawbridges is a bagatelle compared with the errors caused by the other assumptions.

Candidly, the author has very little faith in even the approximate correctness of the ordinary methods of computing live-load stresses in draw-spans; nor has he much more in the superrefined methods involving the principle of least work, or stretching of the different truss members, or the principle of the Three Moments with varying moments of inertia. In his opinion, there is but one satisfactory method of ascertaining the reactions for both balanced and unbalanced loads, viz., by making large models of a number of spans of various lengths, and weighing therewith the reactions for all kinds of loading. From a series of experiments of this kind there could be prepared a diagram or diagrams, similar to that shown on Plate IX, which would give approximately correct reactions for all spans and all loadings. Such an investigation would require considerable time and money; but if some professor of civil engineering would undertake to make the experiments, he could undoubtedly get the models built free of charge by dividing up the work among several of the leading bridge-manufacturing companies. The results of such experiments would be of great value to both the engineering profession and the railroads of America.

In finding dead-load stresses in draw-spans, it is customary to assume that the draw is open. The author follows this method, but also assumes an upward reaction from the lifting machinery at the ends, and finds the stresses therefrom; then, when any such stress tends to increase the section of any member, it is considered, but, when it tends to decrease the section, it is ignored. This method certainly is liable to involve errors on the side of safety; but they will tend to offset some possible errors on the side of danger due to the method employed in finding the live-load stresses.

There will be no attempt made in this treatise to illustrate the detailing of draw-spans and their operating machinery. There is a little work on "The Designing of Draw-spans," by Charles H. Wright, C.E., which attempts to cover this ground, and to which the reader is referred for detailing of drawbridge machinery. Unfortunately, though, the scales used for the drawings are generally too small to make the illustrations satisfactory.

Although, as just stated, the author has no intention of trying to cover here the subject of detailing of the machinery, there are a few details which it will be well for him to touch upon in a general way, among others the question of rim-bearing *versus* centre-bearing turntables. The author is decidedly in favor of the former because of the greater stability involved when the load is carried near the exterior of the pier. Turntables that divide the load between the rim and centre are not to be recommended, because the division is always more or less ambiguous. The load should always be distributed as uniformly as possible over the entire drum and among the rollers, and to do this care should be used in designing the girders over the drum so that they will have not only the necessary strength, but also the proper comparative rigidities. The greater the number of points of support the more evenly will the load be distributed to the drum and rollers, and the deeper the drum the better the distribution. Now as an extra foot of depth of drum costs much less than one foot of height of pivot-pier, it stands to reason that it is always better, whenever practicable, to make the drum much deeper than the calculations for strength and stiffness demand. The only reason for not adopting in every case an excessive depth is that so doing might place the rollers below the level of high water, and thus render the span liable to injury from drift, and the machinery to being blocked by an accumulation of mud under and between the wheels.

When the vertical distance between high water and the lowest part of bottom chord is small, the longitudinal and cross girders can be placed with their bottom flanges flush with the lower surface of the bottom chords, and the drum can be built inside of the box thus formed, so that its bottom flange angles shall be flush with the bottoms of the said girders. Or, if the vertical clearance be great enough to permit it, the box may rest on the drum at either four or eight points.

As a rule, drum diameters are made too great for economy, for many designers think it necessary to rest the tower posts directly over the drum, thus making the diameter of the latter about forty per cent greater than the side of the square, upon the corners of which are located the axes of tower columns. Other designers make the sides of the square intersect the circle of the drum so as to divide the latter into eight equal parts, thus making the diameter of the drum about eight per cent greater than the side of the square.

The author of late years has been taking the diameter of the drum equal to the side of the square, and has obtained eight points of support by inserting four small girders in the corners of the square, at angles of forty-five degrees with its sides. As the cost of a pivot pier varies very nearly as the square of its diameter, it follows that this method of designing the drum effects a great saving in cost of both drum and pier. Occasionally it will give a pier of very small diameter in comparison with the length of the draw-span. The remedy for this, provided the pier have the requisite stability against overturning, is not to increase the pier diameter but to anchor the draw-span to the pier in such a manner as not to interfere with the turning, but so as to offer an effective resistance to any tendency to lift the span off its support.

In the case of the Jefferson City highway bridge, the length of the draw-span is four hundred and forty feet, while the diameter of the drum is twenty-two feet—the same as the perpendicular distance between central planes of trusses. Such a ratio of span length to drum diameter is too great for safety in case of a strong lifting wind acting on one arm only, for such an uplift would have to amount to only twelve and a half pounds per square foot of floor in order to throw the span off the pier. It was therefore necessary to anchor the span to the pier by means of a long four-inch bolt passing through a wide, heavy casting which is embedded in the concrete, and projecting at the upper end between two beams and through a saddle and a heavy washer-plate. The nut on the anchor-bolt is turned down so as nearly but not quite to touch the said washer-plate, thus causing no obstruction to turning the draw, but making the anchorage always ready to resist the slightest tendency to lift the span.

The limiting ratio of length of span to diameter of drum that can be employed without using a central anchorage cannot well be determined by rule, but must always be left to the judgment of the designer. It might suffice, perhaps, to specify that, whenever the uplift on one arm only necessary to upset the draw is less than twenty pounds per square foot of floor in situations exposed to high wind-pressure, or less than fifteen pounds in other situations, an anchorage shall be adopted.

In the case of three draw-spans, which the author has designed lately for the Kansas City, Pittsburg, and Gulf Railroad Company, the span length is two hundred and twenty-five feet, and the diameter of the drum is only seventeen feet; nevertheless no central anchorage was used. In these bridges the open floor reduces the uplift, and the situations are not such that the spans will be exposed to abnormally high wind-pressures.

Heavy draw-spans should be operated by two or more pinions, and when these are placed, as they should be, diagonally opposite each other, some kind of apparatus ought to be used to equalize the pressure on the pinions, otherwise both the latter and the rack are liable to have their teeth broken. The reason for this is that it is impossible to make the toothing of the rack so perfect in the distance of the semi-circumference that opposite pinions operated by a single shaft shall at all times act equally. When electrical machinery is used, the equalizing can be done by means of duplicate motors; but with other machinery some kind of mechanical equalizer should be employed. The author several years ago designed one for the East Omaha draw, which worked to perfection. It was made by cutting the engine-shaft and attaching to each end a bevel-gear wheel. These bevel-gear wheels engage with two small pinions which are inserted between the spokes of a large spur-wheel that turns loosely on the engine-shaft. If we assume the pressures on the main rack-pinions on each side of the drum to be constantly equal to each other, the two halves of the engine-shaft will always have the same angular velocity; but in case the pressure on the teeth of the two rack-pinions on one side of the drum should fall below that on those of the two rack-pinions on the other side, the spur-wheel will move slightly on the shaft until the rack-pinions receive equal pressure again. By this apparatus equal pressure on the teeth of rack and pinions is at all times insured. The author was convinced of the necessity for such a device by watching it when the span was being turned; for several times during each quarter rotation the little pinions on the spur-wheel would make a sudden movement of such magnitude as to indicate a considerable variation in the spacing of the rack-teeth.

In designing draw-spans with high towers, especially long, double-track ones, there is an important matter that is sometimes overlooked, viz., the tendency of the end of the unloaded arm to rise when a moving load is on the other arm. For single-track bridges, the only harm that this would do would be to pound the end bearings; but for a double-track bridge it would certainly some time cause a serious disaster by the derailment of an oncoming train when the other track on the other arm is covered by another train. Before designing the 520-ft. draw-span for the East Omaha bridge, the author looked up this matter as well as he could, having heard of trouble being experienced from rising ends on a double-track draw-span but little shorter than the one then contemplated. The results of the investigation were rather contradictory, so the design was made with three features that were conducive to resist the raising of the ends, viz., extra-deep trusses at both inner and outer hips; stiff, continuously riveted top chords between these points; and an end-lifting apparatus capable of raising the ends one and a half inches. This was the best at that time which the author could do to avoid the difficulty; but at the same time he figured upon using later a holding-down apparatus in case the necessity for same should ever arise. As explained in Chapter XII, this span has at present only a single track at the middle, and the highway cantilevered floors are not yet put on. Observation proves that, with one arm loaded by a train and the other arm empty, there was no rising of the ends when the latter were properly supported. A late inspection showed that the timber cribs, which are used as a temporary support for the ends of the draw, had so shrunk vertically on account of the seasoning of the timber, that the end rollers barely touched their bearings, so the latter will have to be shimmed up. This condition of the ends afforded an excellent opportunity to observe the rise with one arm only loaded by an engine and enough cars to cover the said arm. The amount observed was three eighths of an inch. From this it may be concluded that with masonry piers and the completed superstructure, and with a hoist of one and a half inches by the lifting gear, there is no chance for the ends to rise from their bearings; for, to cause such a rise, it would take a live load just four times as large as the test load, which is more than could be placed on the double-track railway, wagonways, and footwalks. Had the bridge been built with shallow trusses and with eye-bars in a portion of the top chords between outer and inner hips, as was the similar bridge which was reported as giving trouble from rising ends, it is probable that similar difficulty would have been found in this structure.

Some engineers may think that, because each span of a draw is figured as an independent span for unbalanced live loads, on the assumption that the longitudinal tower rods are so small as to carry no vertical shear past the drum, there should be no tendency for the end of one arm to rise when the other arm is loaded; but such is not the case, as the tendency would exist if there were no longitudinal tower rods at all. The rising of one end is evidently due to the lowering of the inner hip of the other span and the consequent pull of the inclined top-chord eye-bars. Now imagine two cables attached to the top of the tower (which is still assumed to be without longitudinal rods), and running to drums on the shore. When these cables are strained sufficiently, the far end of the draw will rise; and under these conditions there will be no vertical shear whatsoever in the tower panel of the truss. As far as vertical shear in this panel is concerned, the conditions for the case of the strained cables and for the single-loaded arm are identical, hence it is proved that one end of a draw can rise when the other arm is loaded and when the longitudinal tower bracing is incapable of carrying any vertical shear past the pivot-pier.

In erecting draw-spans, some method of adjustment must be provided so as to bring the ends to the correct elevation. For comparatively short spans, say up to 200 or even 250 feet, groups of thin plates on top of piers will suffice, as the grade can be adjusted by dapping the ties or joists; but for longer spans there will be needed in addition to this method an adjustment in each of the bottom chords by the insertion of several thin transverse plates in the panels next to the drum.

The tops of all pivot-piers should be so designed as to drain thoroughly by pitching the upper surface from the centre towards the periphery, and by providing at the latter weeping pipes that pass below the lower-track segments.

In large, heavy drawbridges all parts of the turntable and machinery should be made much heavier than the corresponding parts for smaller structures, even if there be no theoretical reason therefor; because the tendency in the past has been to design all portions of the machinery for the exact amounts of work that they are assumed to do, which method gives for many pieces sizes entirely inadequate for some conditions of stress to which they are likely to be subjected. The proportioning of turntables and machinery for draw-spans is a matter involving good judgment and experience in operation rather than intricate mathematical calculations.

The author desires to call attention to the necessity for making all man-power machinery extra strong; because, if there be anything wrong with the apparatus which prevents it from operating properly, the men are liable to crowd upon the levers wherever they can find room, and then surge thereon to their utmost capacity. It was only a short time ago that in operating the East Omaha draw by hand two sets of six or seven men on each of the two four-armed levers failed to start the span in motion. Immediately upon finding the unexpected resistance, they all stepped back a few feet and threw themselves with full force upon the levers, the result being the same as before. The author stopped this instantly, and upon investigation found that the two sets of men were working against each other, so by starting one set in the opposite direction the span was readily put in motion. This example is given to show how ignorant workmen will abuse machinery, and the consequent necessity for making man-power apparatus extra strong, notwithstanding all the opposition that may be offered thereto by bridge manufacturers. It is thought that the method of proportioning such apparatus, which is specified in Chapter XV, will develop ample strength, more especially as the specifications prohibit the use of cast-iron gears.

As a drawbridge is a piece of machinery, it will require a certain amount of care, for otherwise it will get out of order and give trouble just at the wrong time. It should be opened at least once a month, and all parts which move on other parts, especially the wheels and tracks, should be kept clean and well lubricated. The lower rolling surface for the wheels should be kept free from all obstructions, and the wheels should be maintained in proper adjustment by means of the spider-rods. The operating machinery also should receive due care and attention.

CHAPTER XI.

HIGHWAY BRIDGES.

SOME ten years ago the author wrote and published a pamphlet entitled "General Specifications for Highway Bridges of Iron and Steel," the object of which was to effect a much-needed improvement in the designing and building of highway bridges. Through the Engineers' Club of Kansas City, at that time a flourishing society, but now, alas! defunct, the pamphlet was placed in the hands of a great number of bridge engineers throughout the country, with a request that they discuss it for publication. Many of them complied, and their discussions were published in the *Journal* of the Association of Engineering Societies for November 1888. Soon afterwards the first edition of the pamphlet was exhausted, so the author issued a second edition, revised and enlarged, and incorporated therein most of the said discussions. Now, although both editions were circulated widely among county commissioners, and although the author's specifications received the general indorsement of the civil-engineering profession, the effect of the pamphlet on the methods of bridge-building has been practically *nil;* for we continue to read in nearly every issue of *Engineering News* accounts of highway-bridge failures, many of them accompanied by loss of life.

In truth, the number of highway bridge failures is on the increase. This is undoubtedly partially due to the greater number of such structures in existence; but it is also due considerably to the reckless manner in which highway bridges continue to be designed and built, owing to the rapacity of the builders, the ignorance and dishonesty of the commissioners, and the low moral state into which the designers of highway bridges have fallen. So low is that state that, even when given all the metal they could use and a big price for same, it is doubtful whether a single one of them could evolve a structure scientifically designed throughout.

Decidedly, nothing can be done for highway-bridge building through county commissioners, because they are both too ignorant and too corrupt. Nothing but the strong arm of the law will ever reach them; and the only way to force them to build even decently strong structures is to make county commissioners criminally liable for all injuries to persons and pecuniarily liable for all injuries to property due to failures of county bridges built during their tenure of office. Of course, if the commissioners could prove that they had taken all possible precautions by having the structure designed by a specialist of established reputation, built by a good manufacturing company, and inspected by first-class inspectors during both manufacture and erection, they would be able to relieve themselves from the responsibility; but if they were to do all this no bridge failure could occur, or at least the chances for such occurrence would be extremely small. Some such method as this for placing the responsibility for bridge disasters upon county commissioners will be established by law some day, perhaps in the not very distant future; and the sooner the better.

As matters stand now, each new bridge horror stirs up the indignation of the populace, which vows that this time the guilty parties shall be brought to punishment; but the investigation generally drags, personal influence is brought to bear, money is often used judiciously, and the result is that nobody is held responsible, and the disaster is soon forgotten.

If each state were to adopt standard specifications for highway bridges, and if there were a proper officer appointed to see that the counties live up to them, much good would be accomplished.

The second edition of the author's pamphlet on highway bridges is now exhausted, but no third edition will be issued, for the reason that *le jeu n'en vaut pas la chandelle* There will be given, however, in Chapters XVI, XVII, and XVIII of this book complete specifications for the designing of highway bridges of all kinds; consequently this treatise may be said to replace the pamphlet that is now out of print.

There is considerable difference, though, between the old specifications and the new, due to two reasons, viz., first, there have been great advances made in bridge-building in the last eight years; and, second, the author has concluded to abandon the attempt to conciliate those who desire to build cheap structures, so has cut out Class D from his specifications, and has strengthened up and improved the other "classes" in several particulars, notably by raising the minimum thickness of metal from one quarter to five sixteenths of an inch.

The weights of bridges designed according to the new specifications will be somewhat greater than those designed according to the old ones, but the structures will be correspondingly better. Moreover, the new specifications will be found to be more rational, scientific, and generally satisfactory than the old ones, especially in the feature of impact allowance. It must be remembered that the author does not claim that the formula which he specifies for impact will provide exactly for the greatest possible impact on all parts of all highway bridges; but he does think that it will always be great enough, and he knows that any structure in the design of which it is used, and which is proportioned by the specifications of this treatise, will be well and properly designed in every part.

CHAPTER XII.

COMBINED BRIDGES.

As a rule, bridges for carrying both railway and highway traffic are located in or near large cities, although an occasional structure of this kind is found in country districts. The principal advantage of this type of bridge is the saving in first cost, and its principal disadvantage is a reluctance to cross over it on the part of timid drivers, whose horses may be frightened by the trains.

The saving in first cost of a combined railway and highway bridge as compared with two separate bridges for railway and highway traffic is considerable; because the piers for the combined bridge are but little, if any, more expensive than those for the railway bridge, and because the extra metal for the superstructure of the former in comparison with that of the latter is very much less in weight than the weight of metal required for a separate highway bridge.

The prejudice against combined bridges on account of danger is almost wholly unfounded, for horses soon become accustomed to railway trains, and, when screens are employed to hide the latter, but little trouble is experienced on account of frightened horses. These screens may be made either slatted or close, the former offering less resistance to the wind, and the latter being the cheaper.

The advent of the electric railway has somewhat complicated the question of designing combined bridges, for now it is often necessary to accommodate three or four kinds of traffic, viz., railway, electric, wagon, and pedestrian.

When a highway structure has to carry a single-track electric line in addition to the ordinary highway travel, the author classes it simply as a highway bridge, for it is seldom necessary to strengthen it materially, because of the electric-railway load, except in the floor and primary truss members. But if a structure has to carry either a single or double track electric railroad only, the author treats it as a railroad bridge.

Combined bridges may be divided into the following classes:

1. Structures having a single deck for all kinds of traffic, the railway occupying the centre of the bridge, and the electric railway lying close to one truss.

2. Structures having a single-track railway at the middle, a narrow footwalk on each side of same inside of the trusses, and cantilever brackets outside of the latter to carry wagon-ways and electric lines. This arrangement may be varied by running the electric cars over the main railway track, thus leaving the wings free for wagon traffic.

3. Structures having a double-track railway inside of the trusses, with long cantilever brackets outside carrying wagons and electric lines next to the trusses, and pedestrians outside. This arrangement may be varied, as in Case 2, by carrying the electric trains on either one or both of the main railway tracks.

4. Structures having a double-track railway inside of the trusses, with short, cantilever brackets for wagon and electric-railway traffic outside, and either a single passageway overhead at the middle for pedestrians, or two passageways for same on overhead brackets outside of the trusses. As before, this arrangement may be modified by running the electric trains over the main railway tracks.

5. Double-deck structures carrying railway trains on one deck and wagons, electric trains, and pedestrians on the other. The pedestrians may be accommodated either inside the trusses or, preferably, by exterior walks on cantilever brackets. The railway may be placed either above or below to suit the existing conditions, or there may be either a single-track or a double-track railway both above and below, with wagon-ways and pedestrian-ways outside of the trusses either above or below.

Class No. 1 is the cheapest possible kind of combined bridge, and at the same time the most unsatisfactory, for when a railroad train is about to pass over the bridge all wagon and electric-railway travel must be kept off, and because pedestrians must look out sharply for their safety when on the structure with a railway train crossing. Their danger is really greater, though, when an electric train is passing a team or teams. The least allowable clear width of bridge for this class of structure is twenty feet, the electric cars running on a third rail and on one of the rails of the main railway. The author has built a large bridge of this class, and it has never given any trouble from the combined traffic, which, however, is, up to the present, rather light.

Class No. 2 is a very satisfactory type of structure. The author has designed and built several bridges of this kind, the largest of which is the Combination Bridge Company's bridge over the Missouri River at Sioux City, Iowa. It consists of two draw-spans of 470 feet each and two fixed spans of 500 feet each, the distance between central planes of trusses being twenty-five feet.

Class No. 3 is also a satisfactory type of structure. The author has built a large bridge of this class, viz., the one across the Missouri River at East Omaha, Nebraska. This class of structure involves very heavy metal-work; but it is not uneconomical.

Class No. 4 is an unusual type, and is not likely to be called for very often, although the author once had occasion to figure on a bridge of this kind.

Class No. 5 is a very good combination that can be modified to suit nearly any conditions of combined traffic.

A good example of this is the design described in Chapter IX for the Kansas City and Atlantic Railway Company's proposed bridge over the Missouri River at Kansas City, Mo.

The East Omaha Bridge just referred to affords an excellent example of how to keep down the first cost of a structure and yet build it so that it can be enlarged later after the business develops. The design for the final structure involves a draw-span of 520 feet and a fixed span of 560 feet, carrying a double track railway between the trusses, a combined wagonway and electric railway outside of each truss, and a pedestrian-way outside of each wagon-way, the bridge crossing the river at right angles; while the present structure consists of the 520-foot draw-span, without the wings, and three single-track combination spans of 192 feet each, all the piers except the pivot-pier being built of piles and timber, and the centre line of structure making an angle of eleven degrees with the centre line of the final bridge. The deck carries a single railway track at the middle and an electric line by means of a third rail to one side. All four classes of travel use this deck. The only portion of the existing structure that is really finished is the pivot-pier, which consists of a double steel cylinder forty feet in diameter sunk by open dredging to bed-rock, which lies one hundred and twenty-two feet below extreme low water. The completion of the draw-span will be a very simple matter, consisting merely of adding the cantilever brackets with their stringers and flooring and laying the electric-railway rails thereon. The remaining piers for the final structure can all be put in, and the fixed span can then be placed on them without interrupting traffic, because of the deflection downstream of the present temporary structure. When the new bridge is completed, all that it will be necessary to do is to rotate the draw eleven degrees, so that the traffic may be transferred thereto. Afterwards the old pile piers and the combination spans can be removed at pleasure.

In designing combined bridges of all classes except No. 1, a considerable economy of metal may be effected legitimately by keeping the total live load as low as is proper with reference to the theory of probabilities. For instance, in Class No. 2 the live load for trusses can be determined by adding to the equivalent uniform live load, given in the diagram on Plate III or Plate IV, a much smaller highway floor load per lineal foot of span than that prescribed in the specifications for highway bridges, because when the greatest train load is on the bridge, the chances of having a heavy highway live load are very small. The longer the span the smaller may the live load per square foot of floor be taken when finding the total live load for the trusses.

Again, in Classes No. 3 and No. 4 it would be legitimate to take the live load per lineal foot for the railway equal to twice the car load per lineal foot, and add thereto a small highway live load as in the last case.

Finally, in Class No. 5 it would be proper for a four-track bridge to make the live load for the trusses equal to four times the car load per lineal foot, and ignore entirely the highway live load; for the greatest combined live load would never amount to four times the car load. This was the method pursued by the author in determining the live load for the trusses of the proposed Kansas City and Atlantic Railway Company's bridge referred to in Chapter IX.

CHAPTER XIII.

DETAILING.

It is only within a few years that much attention has been given to detailing by bridge engineers, the old custom having been for the engineer to figure the diagram of stresses, or, as it was then called, the strain-sheet, and pass it over to a draftsman (too often a cheap one) to make therefrom the working drawings of the bridge, using probably some old drawings of another bridge as a guide for the detailing. Concerning the evil effects of such a course of action the engineer who does much inspection of existing structures can speak authoritatively. If questioned upon the subject, any such engineer will say that nearly all bridges which fail or which are condemned and removed, are deficient in strength of details rather than in strength of main members.

Some years ago the author had occasion to examine and report upon nearly all the bridges on two hundred miles of the main line of an important Western road, with the result that he found it necessary to condemn almost all of them. A few have since been repaired, but most of them have been taken out and replaced. In most of these condemned structures the detailing was so faulty that the bridges were gradually racking to pieces, and no amount of patchwork would have made them really serviceable. It is true that the main members were considerably overstrained by reason of the increase in rolling loads, but had the details been first-class the structures would have been standing to-day.

Just here it may be well to mention that the inspection of these bridges caused the author to establish for himself the following principle, which, as it does not pertain to bridge-designing, is not given in Chapter II: "In nine cases out of ten the proper way to strengthen a weak bridge is to take it out and replace it with a good one, throwing the old metal into the scrap heap."

In railway-bridge designing, for a number of years, the average ratio of weight of details to weight of main members has been gradually increasing; and the end is not yet, because the average bridge-designer has still a great deal to learn concerning the importance of good and efficient detailing. As long as contracts for bridges are awarded to bridge companies on competitive designs, and the structures are paid for by the lump sum instead of by the pound, just so long will the science of detailing be ignored, and just so long will bridges be built which will eventually wear out, simply for want of a little more metal distributed just where it is needed, viz., in the details.

The author feels that he cannot speak too forcibly concerning the importance of thoroughly scientific detailing for all kinds of metal-work; for what avails it that a structure have an excess of section in every main member, if a single important detail be lacking in strength? If the author were in a position where he had to cut down the weight of a structure even as much as thirty per cent, he would unhesitatingly take the metal almost entirely out of the sections of the main members and leave the detailing practically unchanged. A structure thus designed would long outlast one of the same type in which the weight of the details and that of the main members were reduced in the same proportion.

A few years ago the standard text-books on bridges ignored entirely the subject of detailing. Later they have taken cognizance of it by illustrating certain details in common use, both good and bad (generally the latter), but have failed to state the fundamental principles that should govern the designing of all details. These general underlying principles and complete instructions as to how to detail scientifically the author has endeavored to give in Chapters II, XIV, XV, XVI, and XVII of this work. The bridge designer, by studying these chapters carefully, mastering all of their contents, and, while making his drawings, applying the principles therein given, will be able to evolve structures that, to say the least, will be a great improvement on the average structure in common use.

CHAPTER ~~XVI~~ XIV.

GENERAL SPECIFICATIONS GOVERNING THE DESIGNING OF STEEL RAILROAD BRIDGES AND VIADUCTS AND THE SUPERSTRUCTURE OF ELEVATED RAILROADS.

GENERAL DESCRIPTION.

MATERIALS.

ALL parts of the structure, except ties, foot-planks, and guard-timbers, shall, for all spans of ordinary lengths, be of medium steel, excepting only that rivets and bolts are to be of soft steel, and adjustable members of either soft steel or wrought iron. For very long spans high steel may be used for top chords, inclined end posts, pins, eye-bars in bottom chords, and those in main diagonals of panels where there is no reversion of stress when impact is included. It may be used also for the web-members of cantilever and anchor arms in cantilever bridges where the variation of stress is comparatively small and where the impact cannot be great. Excepting for purely ornamental work, cast iron will not be allowed to be used in the superstructure of any bridge, trestle, or elevated railroad, cast steel being employed wherever important castings are necessary.

CROSS-TIES, FOOT-PLANKS, AND GUARD-TIMBERS.

Cross-ties, foot-planks, and guard-timbers shall be of long-leaf, Southern yellow pine or other timber which, in the opinion of the Engineer, is equally good and serviceable. The wooden floor shall be so designed as to ensure safety from passing trains for the railroad employees. The spaces between ties shall, in general, not be less than five (5) inches nor more than six (6) inches wide. The sizes of ties shall be such as to give the requisite resistance to bending, under the assumption that the load on one pair of wheels is distributed equally over three ties, the effect of impact being considered. No tie shall be less than seven (7) or preferably eight (8) inches wide, nor less than six (6) inches deep, nor less than ten (10) feet long, except in the case of elevated railroads, where the length may be reduced to eight (8) feet for a spacing of five (5) feet between central planes of longitudinal girders.

Ties shall be dapped to a full and even bearing not less than one-half (½) inch onto the stringers; and each alternate tie shall be secured thereto at each end by a three-quarter (¾) inch hook bolt.

All timber bolts shall be of soft steel, with cold-pressed threads.

Outer guard-timbers shall be 6″ × 8″ laid on flat, dapped one (1) inch onto the ties, and placed so that their inner faces shall be just twelve (12) inches from the gauge-planes of rails.

Where inner guard-timbers are employed, they shall be 6″ × 8″ on flat, dapped one (1) inch onto the ties, and placed so that their outer faces shall be just five (5) inches from the gauge-planes of rails.

Each guard-rail must be bolted to each alternate tie by a three-quarter (¾) inch screw-bolt, the head of which shall be countersunk into the wood by means of a cup-shaped washer. Each guard-timber must be spliced over a tie with a half-and-half joint of at least six (6) inches lap, through which must pass a three-quarter (¾) inch screw-bolt.

Guard-timbers shall extend over all piers and abutments.

Steel rails or heavy steel angles well fastened to the ties may be substituted for the inner wooden guard-rails, or the inner guards may be omitted altogether if the Engineer so direct.

RERAILING APPARATUS.

At each end of every bridge or trestle, there is to be placed a rerailing apparatus that will, in the most effective manner practicable, return to the track any derailed car or locomotive that is not more than half the width of track gauge out of line.

BUCKLED-PLATE FLOORS.

If the Engineer so desire, a buckled-plate floor with ties in ballast may be used instead of the wooden floor, in which case the size of the ties may be reduced to $6'' \times 8'' \times 8'$.

All buckled-plate floors must be thoroughly drained so as not to retain water, and the upper surface of the buckled plate must be protected from rusting by a liberal use of the best obtainable preservative coating.

SUPERELEVATION ON CURVES.

On curves the outer rail will be elevated the proper amount for the degree of curvature and for the assumed medium velocity of trains; and this elevation must be framed into ties, as no shims will be allowable anywhere under ties or rails, excepting in the case of very sharp curves requiring a superelevation exceeding three (3) inches in five (5) feet, on which long shimming timbers are to be bolted to the top flanges of the outer longitudinal girders, or short, substantial ones to tops of ties, so as to give the required superelevation.

The formula to be used for total superelevation on standard-gauge roads is

$$E = \frac{0.3277\, V^2}{R},$$

where E is the total superelevation in feet of the exterior rail above the interior rail, V is the assumed velocity of train in miles per hour, and R is the radius of the curve in feet. The total superelevation is to be obtained by depressing the inner rail and elevating the outer one equal amounts, thus preserving the grade of the centre line.

SPACING OF STRINGERS, GIRDERS, AND TRACKS.

In general, stringers for through bridges shall be spaced eight (8) feet centres for single-track bridges and six (6) feet six (6) inches for double-track bridges and half-through plate-girder bridges. In elevated railroads the spacing of the longitudinal girders may be made as small as five (5) feet centres.

Deck plate-girders may be spaced from six (6) feet to ten (10) feet centres, the usual distance being the nearest even foot to one tenth ($\frac{1}{10}$) of the span; but in high trestles the spacing shall, preferably, be ten (10) feet, and never less than eight (8) feet.

The standard distance between centres of tracks on tangent for surface railroads shall be thirteen (13) feet, while for elevated railroads it shall generally be twelve (12) feet.

SPACING OF TRUSSES.

From centre to centre of through-trusses the perpendicular distance shall not be less than seventeen (17) feet, or one twentieth ($\frac{1}{20}$) of the span length.

From centre to centre of deck, pin-connected, or riveted trusses the perpendicular distance shall not be less than ten (10) feet or one thirteenth ($\frac{1}{13}$) of the span length, except in the case of elevated railroads, where open-webbed, riveted girders are adopted. These may be spaced according to the directions given for plate girders.

CLEARANCES.

The clear opening on tangent shall not be less than that shown in Fig. 7.

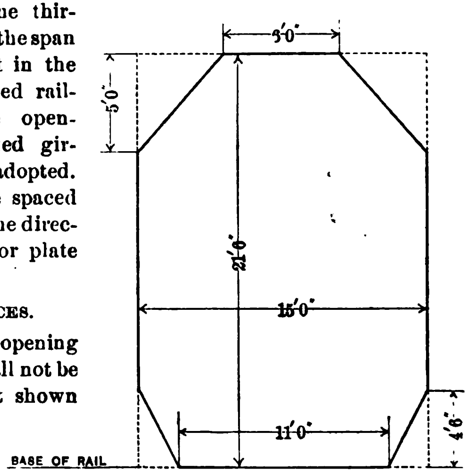


FIG. 7.

On curved track, the horizontal distance from the centre of track to clearance line shall be increased at all points two (2) inches for each degree of curvature.

EFFECTIVE LENGTHS.

Effective lengths shall be as follows:

For pin-connected spans, the effective length shall be the distance between centres of end-pins of trusses.

For riveted girders, it shall be the distance between centres of bearing-plates.

For stringers, it shall be the distance between centres of cross-girder webs.

For cross-girders, it shall be the perpendicular distance between central planes of trusses.

For columns and posts, it shall be the greatest length between points of axis that are rigidly held in the direction in which the strength is being considered.

These effective lengths are to be used in calculating moments, stresses, and working strengths.

EFFECTIVE DEPTHS.

Effective depths shall be as follows:

For pin-connected trusses, the perpendicular distance between gravity lines of chords, which lines must pass through centres of pins.

For plate-girders and open-webbed riveted girders, the perpendicular distance between centre lines of gravity of upper and lower flanges; but never to exceed the depth from out to out of flange angles.

STYLES OF BRIDGES FOR VARIOUS SPAN LENGTHS.

For spans under fifteen (15) feet, rolled I beams.

For spans between fifteen (15) feet and eighty-five (85) feet, plate girders.

For spans between eighty-five (85) feet and one hundred and twenty-five (125) feet, "A" truss, pin-connected spans, or riveted, open-webbed girders of single cancellation.

For spans between one hundred and twenty-five (125) feet and one hundred and seventy-five (175) feet, riveted, open-webbed girders of single cancellation, or pin-connected trusses designed with special reference to extreme rigidity in all parts.

For spans exceeding one hundred and seventy-five (175) feet, pin-connected spans.

The use of pony-truss bridges of any kind is prohibited, excepting only half-through, plate-girder spans, in which the top flanges are held rigidly in place by brackets riveted to cross-girders that are spaced generally not to exceed fifteen (15) feet apart.

In general, double-track bridges shall have only two trusses, in order to avoid spreading the tracks.

FORMS OF TRUSSES.

The forms of trusses to be used are as follows:

For pin-connected spans up to one hundred and twenty-five (125) feet, the "A" truss.

For open-webbed, riveted girders, the Warren or triangular girder with verticals dividing the panels of the top chords; also the Pratt truss.

For deck-spans having top chords supporting wooden ties, the Warren or triangular girder with verticals dividing the panels of the top chords.

For spans between one hundred and twenty-five (125) feet and about two hundred and fifty (250) feet, Pratt trusses with top chords either straight or polygonal,

For spans exceeding two hundred and fifty (250) feet, Petit trusses.

It is understood that these limiting lengths are not fixed absolutely, as the best limits will vary somewhat with the number of tracks and weight of trains.

MAIN MEMBERS OF TRUSS-BRIDGES.

All spans of every kind shall have end floor-beams, riveted rigidly to the trusses or girders, for supporting the stringers.

Stringers are to be riveted to the webs of the cross-girders.

In general, all trusses shall have main end posts inclined.

All trusses shall be so designed as to admit of accurate calculation of all stresses, excepting only such unimportant cases of ambiguity as that involved by using two stiff diagonals in a middle panel.

All lateral bracing and other sway-bracing shall be rigid both above and below, i.e., the sections must be capable of resisting compression, adjustable rods for such bracing being allowed only in towers of draw-spans and in lower lateral systems of deck-bridges.

The stiff diagonals of lower lateral systems, which shall be of double cancellation, shall be riveted rigidly to the stringers where they cross them, so as to transfer in an effective manner the thrust of braked trains to the truss-posts without causing a horizontal bending on the cross-girders.

All through-spans shall have stiff portal bracing at each end, connected rigidly to the inclined end posts. The said portal bracing shall be made as deep as the specified clear headroom will allow.

When the height of the trusses is great enough to permit it, there shall be used at each panel point a rigid bracing frame riveted to the top lateral strut and to the posts, and carried down to the clearance line. When the truss depth is not great enough for this detail, corner brackets of proper size, strength, and rigidity are to be riveted between the posts and the upper lateral struts.

Deck-bridges shall have stiff, diagonal braces between opposite vertical posts, which bracing, as a matter of precaution, shall have sufficient strength to carry one half of a panel-truss live load with its impact allowance; and the transverse bracing between the vertical or inclined posts at each end shall be sufficiently strong to transmit properly to the masonry one half of the wind-pressure and centrifugal load (if there be any) which is carried by the entire upper lateral system of the span.

The lower lateral systems of deck-bridges shall be made of adjustable rods in alternate panels, thus leaving every other panel unbraced, and forcing the wind-pressure from below up the vertical bracing and to the ends of the span by the upper lateral system.

Suspenders or hip verticals and two or more panel lengths of bottom chord at each end of each span shall, preferably, be made rigid members, excepting that in "A" trusses the bottom chords and centre verticals are to be of eye-bars.

All floor-beams are to be riveted to the truss-posts in truss-spans, excepting in the case of Petit trusses when the suspenders are of eye-bars. In these, floor-beam hangers may be used, provided they be made of plates or shapes, and that they be stayed at their upper ends against all possibility of rotation.

CONTINUOUS SPANS.

Except in the case of swing-bridges or cantilevers, consecutive spans are not to be made continuous over the points of support.

TRESTLE TOWERS.

As a general rule, each trestle-bent shall be composed of two columns battered from one and a half ($1\frac{1}{2}$) to two and a half ($2\frac{1}{2}$) inches to the foot, the bents being united in pairs to form towers. Each tower thus formed shall be thoroughly braced with rigid bracing on all four faces, and shall have four horizontal struts at the base. In each intermediate horizontal plane of division, formed by the panels of the tower bracing, there is to be a pair of diagonal adjustable rods to bring the columns into proper position and to retain them there.

The feet of the columns must be attached to anchorages capable of resisting twice the greatest possible uplifting; and the details of the metal-work connecting the anchor-rods to the columns must be such as to make the metal-work and pedestals act as a single piece, so that, if tested to destruction by overturning, the bent would not fail between the superstructure and the substructure.

While it is desirable to have sufficient base to prevent any tension from coming on the anchor-bolts, it is not advisable on this account to make the batter of the columns too great, especially in very high trestles.

When trestle-bents become unduly wide, a vertical column is to be placed midway between the legs so as to divide up the transverse and horizontal sway-bracing.

Care must be taken to provide properly for expansion and contraction at column feet both transversely and longitudinally.

In elevated railroads, the towers can be placed at about every fourth span or, say, every one hundred and fifty feet, or can be dispensed with altogether, when the conditions so require, by strengthening the columns properly to resist traction, thrust of braked trains, and the longitudinal component of diagonal wind-pressure.

ADJUSTABLE MEMBERS.

It is preferable to avoid altogether the use of adjustable members in trusses, as well as in sway-bracing. If the structure must be made as cheap as possible, adjustable counters may be employed; but it is advisable to confine their use, as before stated, to diagonals in towers of swing-spans and in lower lateral systems of deck-bridges.

CAMBER.

All trusses must be provided with such a camber that, with the heaviest live load on the span, the total camber shall never be quite taken out by deflection. With parallel chords, sufficient camber will be obtained by making the top-chord sections longer than the corresponding bottom-chord sections by one eighth ($\frac{1}{8}$) of an inch for each ten (10) feet of length. One half of the camber after a span is swung is to be taken out of the track by dapping the ties, unless this would cut too deeply into the timber.

Plate girders and shallow, open-webbed, riveted girders should not be given any camber.

EXPANSION.

Every span must be provided with some means of longitudinal expansion and contraction due to changes of temperature over a range of one hundred and fifty (150) degrees Fahrenheit.

Spans up to eighty-five (85) feet in length, or in certain cases up to even one hundred (100) feet, may slide on planed surfaces; but those of greater length must move on nests of turned rollers. Occasionally a rocker end is permissible; but this method of expansion is always to be avoided if practicable.

ANCHORAGE.

Every span must be anchored at each end to the pier or abutment in such a manner as to prevent the slightest lateral motion, but so as not to interfere with the longitudinal motion of the trusses or girders due to changes of temperature or loading.

NAME-PLATES.

The names of the designer, manufacturer, and builder of every bridge or trestle, also the date of erection, must be attached thereto in a prominent position and in a durable manner.

LOADS.

The loads to be considered in designing bridges, trestles, and elevated railroads are the following; and all parts of same are to be proportioned to sustain properly the greatest stresses produced thereby for all possible combinations of the various loads.

A. Live Load.
B. Impact Allowance Load.
C. Dead Load.
D. Direct Wind Load.
E. Indirect Wind Load, or Transferred Load.
F. Traction Load.
G. Centrifugal Load.
H. Effects of Changes of Temperature.

In calculating the stresses caused by a uniform moving load, the load shall be assumed to cover the panel in advance of the panel point considered; but the half-panel load going to the forward panel point will be ignored; or, in other words, the uniform load will be treated as if concentrated at the various panel points.

In deck-spans on sharp curves, after the centre curve for each rail and the centre lines of the longitudinal girders are laid out, the approximate extra live load on the outer girder due to the projection of the curve of the rail beyond its centre line near mid-span is to be computed and added to the regular live load; but the corresponding excess of dead-load from the flooring, being small, is to be ignored. As the superelevation provides for an equal distribution of the live load on the rails for the assumed medium velocity of trains, there will be an excess of live load on the outer girder due to the velocity being sometimes greater than this; but the said excess is so small that it is to be ignored.

The excess of live load on the inner girder, due to the velocity of train being sometimes less than that assumed for determining the superelevation, is offset by the reduced load due to the projection of the centre line of the rail near mid-span beyond the centre line of the girder; so it also is to be ignored.

LIVE LOADS.

The live load to be used in designing any railroad structure shall be taken from the "Compromise Standard System of Live Loads for Railway Bridges and the Equivalents for Same," which is given in Chapter XIX and in Plates I, II, III, and IV.

In single-track bridges but one of the seven classes of loading given can be used for any span; but in bridges having more than one track two or even three classes of loading can be used in the same span, if so desired by the Engineer: for instance, Class W could be adopted for stringers, Class X for cross-girders, and Class Y for trusses, thus utilizing the theory of probabilities.

The equivalent live loads given on the diagrams are to be used instead of the actual wheel concentrations.

For elevated railroads the live loads are generally to be very much lighter than that of Class Z of the Compromise Standard System; but the said loads will have to be determined for each individual system of elevated railroad, so as to provide for the greatest train load that can ever come upon the structure, but for no more.

IMPACT ALLOWANCE LOAD.

The impact allowance load is to be a percentage of the equivalent uniform live load, found by the formula

$$P = \frac{40000}{L + 500},$$

where P is the percentage and L the length in feet of span or portion of span that is covered by the live load, when the member considered is subjected to its maximum stress.

DEAD LOAD.

The dead load is to include the weight of all the metal and wood in the structure, excepting that of those portions resting directly on the abutments, whose weights do not affect the stresses in the trusses; also any other permanent load that may be carried by the structure.

The following unit weights are to be assumed in estimating the dead load:

Creosoted lumber four and one-half ($4\frac{1}{2}$) pounds per foot board measure.

Oak and other hard woods four and a quarter ($4\frac{1}{4}$) pounds per foot board measure.

Yellow pine three and three-quarters ($3\frac{3}{4}$) pounds per foot board measure.

White pine and other soft woods two and three-quarters ($2\frac{3}{4}$) pounds per foot board measure.

Rails and their fastenings, sixty (60) pounds per lineal foot per track.

Two thirds ($\frac{2}{3}$) of the dead load shall be assumed to be concentrated at the panel points of the lower chords in through-bridges and at those of the upper chords in deck-bridges; and one third ($\frac{1}{3}$) of the dead load at the panel points of the upper chords in through-bridges and at those of the lower chords in deck-bridges.

If in any bridge design the dead load assumed should differ from that computed from the diagram of sections and the detail drawings by an amount exceeding one (1) per cent of the sum of the equivalent live load and actual dead load, the calculations of stresses, etc., are to be made over with a new assumed dead load.

WIND LOADS.

For railroad bridges the wind loads per lineal foot of span for both the loaded and the unloaded chords are to be taken from the curves given in Plate VII.

The wind loads for the loaded chords include a pressure of three hundred (300) pounds per lineal foot on the train, the centre of which pressure is applied at a height of eight (8) feet above the base of rail.

For determining the requisite anchorage for a loaded structure, the train of empty cars shall be assumed to weigh one thousand (1000) pounds per lineal foot.

In trestle towers the columns and transverse bracing shall be proportioned to resist the following wind-pressures in addition to all other loads.

1st. When the structure is loaded, four hundred and fifty (450) pounds per lineal foot on stringers and cars, and two hundred and fifty (250) pounds for each vertical foot of each entire tower.

2d. When the structure is empty, three hundred and fifty (350) pounds per lineal foot on stringers, assumed to be concentrated one foot above the centre of stringer, and three hundred and fifty (350) pounds for each vertical foot of each entire tower.

The wind loads for longitudinal bracing are to be taken as seven tenths (0.7) of those for the transverse bracing.

In figuring greatest tension on columns and anchor-bolts, computations are to be made for both the loaded and the unloaded structure, in double-track trestles placing the train of empty cars on the leeward track.

All wind loads are to be treated as *moving loads*

INDIRECT WIND LOAD OR TRANSFERRED LOAD.

For both through and deck spans, even with polygonal top chords, the transferred load is to be assumed to produce a tension in the leeward bottom chord that is constant from end to end of span, and a similar release of tension on the windward bottom chord. For trusses with parallel chords this assumption is correct, provided that all the wind-pressure travels directly to ends of span by the horizontal bracing; while for trusses with polygonal top chords the assumption is a compromise, the travel of wind-pressure being ambiguous. The transferred load at one pedestal is to be found by multiplying one half of the total wind load on the top chord by the average truss depth and dividing the product by the perpendicular distance between central planes of trusses.

TRACTION LOAD.

The total traction load on any portion of a structure is to be taken as twenty (20) per cent of the greatest live load that can be placed on that portion of said structure.

In proportioning the towers and columns of trestles and elevated railroads, the towers and columns between consecutive expansion points are to be assumed to receive no aid from neighboring towers and columns, but must be figured for the greatest possible traction load between said consecutive expansion points.

No percentage of impact is to be added to traction loads.

CENTRIFUGAL LOAD.

The centrifugal load is to be computed for the greatest probable velocity of trains by the formula

$$C = \frac{wv^2}{32.2R},$$

where C is the centrifugal load per lineal foot, w is the equivalent live load per lineal foot, v is the velocity of train in feet per second, and R is the radius of the curve in feet.

All portions of the structure affected by the centrifugal load are to be figured to carry properly the stresses induced by the said load in addition to all other stresses to which they may be subjected.

No percentage of impact is to be added to centrifugal loads.

EFFECTS OF CHANGES OF TEMPERATURE.

In ordinary structures changes of temperature will not affect the stresses in the members, provided, of course, that proper precaution be taken to permit unrestricted expansion and contraction. But in all arches, excepting only those hinged at both ends and at the crown, the stresses caused by the assumed extreme changes of temperature must be computed and duly considered.

INTENSITIES OF WORKING-STRESSES.

The following intensities of working-stresses (i.e., pounds per square inch of cross-section) are to be used for all cases, except where wind loads are combined with other loads, under which conditions the said intensities are to be increased thirty (30) per cent. But when high steel is employed the metal is to be strained fifteen (15) per cent higher for all cases than herein specified, even after the said thirty (30) per cent has been added to allow for wind stresses.

Tension on eye-bars in bottom chords and main diagonals, and on lateral rods........	18,000 pounds.
Tension on shapes in bottom chords, main diagonals and laterals, on eye-bars in suspenders and hip verticals, and on soft-steel adjustable truss members..................	16,000 "
Tension on net section of plate-girder flanges (assuming one eighth of the area of the web to act as a part of each flange), extreme fibres	

of rolled I beams, and on shapes in body of suspenders, hip verticals and hanger-plates (there being 50 per cent increase of net area for section through eyes)	14,000 pounds
Tension on adjustable truss members of wrought iron	13,000 "
Bending on pins	27,000 "
Bearing on pins and rivets (measured upon the projection of the semi-intrados upon a diametral plane)	22,000 "
Shear on pins and rivets	12,000 "
Shear on webs of plate girders	10,000 "

For field-rivets the intensities for bearing and shear are to be reduced twenty-five (25) per cent.

Compression on top chords.......... $18,000 - 70\dfrac{l}{r}$;

Compression on inclined end posts... $18,000 - 80\dfrac{l}{r}$;

Compression on all other struts with fixed ends........................ $16,000 - 60\dfrac{l}{r}$;

Compression on all other struts with one or two hinged ends........... $16,000 - 80\dfrac{l}{r}$;

where l is the unsupported length of the strut in inches and r is its least radius of gyration in inches.

Compression on end stiffeners of plate girders.	14,000 pounds.
Tension on extreme fibres of long leaf, Southern, yellow-pine timber in bending, the effect of impact being considered	2,000 "

BEARINGS UPON MASONRY.

All bed-plates must be of such dimensions that the greatest pressures on the masonry, including impact, shall not exceed those given in the following table:

Material.	Permissible Pressure per Square Inch.
Am. Nat. Cement Concrete	130 pounds.
Brickwork laid in Cement	170 "
Portland Cement Concrete	200 "
Ordinarily Good Sandstone	200 "
Extra Good Sandstone	250 "
Yellow Pine or Oak on Flat	300 "
Ordinarily Good Limestone	300 "
Extra Good Limestone	400 "
Granitoid	450 "
Granite	550 "

REVERSING-STRESSES.

In case stresses reverse, the areas required for both tension and compression, including impact in each case, are to be figured separately, and three fourths ($\frac{3}{4}$) of the smaller area is to be added to the larger area in order to obtain the total sectional area of the piece. The rivets, however, are to be figured for the sum of the two stresses, both impacts included.

The effect of reversion of stresses in case of wind loads is to be ignored when computing sectional areas of members and the number of rivets required; but, of course, wherever reversion of stress occurs, the piece must be stiffened so as to resist compression.

NET SECTION.

The net section of any tension flange or member shall be determined by a plane cutting the member square across at any point. The greatest number of rivet-holes which can be cut by any such plane, or whose centres come nearer than two and a half ($2\frac{1}{2}$) inches to said plane, are to be deducted from the gross section when computing the net area.

BENDING MOMENTS ON PINS.

In figuring the bending moments on pins, the stresses shall be assumed as concentrated at centres of bearings.

COMBINATIONS OF STRESSES.

In the girders of plate-girder spans and of deck, open-webbed, riveted-girder spans, the only stresses that need to be considered are those caused by the live, impact, dead, and centrifugal loads.

The trusses of through-bridges will be affected by the live, impact, dead, direct wind, and indirect wind loads; and in exceptional cases also by the centrifugal load. The trusses of deck-bridges will be affected by all of these loads. In no case will the traction load affect the trusses of bridges to such an extent as to require consideration; consequently the only provision for traction load required in through and deck bridges is adequate rigid bracing to carry it from the track to the trusses without subjecting any portion of the structure to an improper loading, as, for instance, the flanges of cross-girders to horizontal bending.

In *bridges* of all kinds the various loads herein specified shall be combined without any reduction; but in *trestles*, more especially very high ones, it will be legitimate, when combining the stresses from the various loadings, to reduce some of them or even to ignore some entirely, in order to avoid proportioning for any highly improbable or impossible combination of loads. For instance, when a trestle is situated near the middle of a sharp curve or near the apex of two heavy rising grades, it would be incorrect to assume a high velocity of train. In such cases as these the element of individual judgment in combining the stresses from the various loads and in assuming the sizes of the latter cannot well be eliminated.

BENDING ON TOP CHORDS.

For combined direct stresses and bending on chords, the moment is to be computed by the compromise formula

$$M = \frac{1}{10} Wl,$$

where W is the total transverse load in pounds on the piece, including impact, and l is the length of the piece in inches.

The extreme fibre-stress for the combination shall not exceed sixteen thousand (16,000) pounds; and the moment at mid-panel is to be assumed the same in amount as that at the panel points.

Top chords subjected to transverse loading should be made as deep as economy of metal will permit.

BENDING ON INCLINED END POSTS.

In proportioning inclined end posts of trusses of through-bridges for a combination of all the loads herein specified, together with the bending caused by the wind-pressure which travels transversely down the piece to the pier or abutment, the extreme fibre may be strained thirty (30) per cent higher than the intensity specified for the direct compression, the bending moment being computed on the assumption that the inclined end post is fixed above by the portal bracing and at the bottom by its connections to the pedestal and end floor-beam, thus making the lever-arm of the moment equal to one half the length of that portion of the inclined end post lying between the centre of pedestal-pin and the centre of the lower portal strut (or, in case of plate-girder portals, the bottom of the said plate girder).

BENDING DUE TO WEIGHT OF MEMBER.

If the extreme fibre-stress resulting from the bending due to the weight only of any member does not exceed ten (10) per cent of the specified intensity of working-stress, the effect of such bending may be ignored; but, if it does so exceed, its effect must be combined with those of the other stresses, using, however, for determining the sectional area, an intensity of working-stress ten (10) per cent greater than that specified.

GENERAL LIMITS IN DESIGNING.

The following general limits shall be adhered to in designing bridges, trestles, viaducts, and the line-work of elevated railroads:

No metal less than three-eighths ($\frac{3}{8}$) of an inch in thickness shall be used except for filling-plates.

The least allowable thicknesses of webs of rolled I beams shall be as follows:

24″ I beams	$\frac{5}{8}$″	webs.
20″ "	$\frac{9}{16}$″	"
18″ "	$\frac{1}{2}$″	"
15″ "	$\frac{7}{16}$″	"
12″ "	$\frac{3}{8}$″	"

No channel less than ten (10) inches in depth shall be used, except for lateral struts, in which eight (8) inch channels may be employed.

No angles less than $3'' \times 2\frac{1}{2}'' \times \frac{3}{8}''$ shall be used, except for lacing.

No eye-bars less than four (4) inches deep or three quarters ($\frac{3}{4}$) of an inch thick shall be employed; and the depths of eye-bars for chords and main diagonals shall not be less than one fifty-fifth ($\frac{1}{55}$) of the length of the horizontal projection of same.

No adjustable rod shall have less than one square inch of cross-section.

The shortest span length for trusses with polygonal top chords shall be one hundred and seventy-five (175) feet.

The limit of span length in which the stringers can be riveted continuously from end to end of span shall be two hundred (200) feet. Beyond this limit sliding bearings must be used at one or more intermediate panel points; and in no span shall there be a length of continuously riveted stringers exceeding two hundred (200) feet.

For all compression-members of trusses and for columns of viaducts and elevated railroads the greatest ratio of unsupported length to least radius of gyration shall be one hundred (100), excepting those members whose main function is to resist tension. In these the limit may be raised to one hundred and twenty (120).

The corresponding limit for all struts belonging to sway-bracing shall be one hundred and forty (140).

GENERAL PRINCIPLES IN DESIGNING ALL STRUCTURES.

In designing all structural metal-work the following principles are invariably to be observed:

1. All members must be straight between panel-points, as curved struts or ties will under no circumstances be allowed.

2. The axes of all members of trusses or girders and those of lateral systems coming together at any apex of a truss or girder must intersect at a point, whenever such an arrangement is practicable; otherwise the greatest care must be employed to ensure that all the induced stresses and bending moments caused by the eccentricity be properly provided for.

3. Truss members and portions of truss members must always be arranged in pairs symmetrically about the central plane of the truss, except in the case of single members, the axes of which lie in said central plane of truss. This applies also to the designing of open-webbed, riveted girders.

4. In proportioning main members of bridges, symmetry of section about two principal planes at right angles to each other is to be attained wherever practicable; but in designing top chords and inclined end posts this rule cannot be followed.

5. In both tension and compression members, the centre line of applied stress must invariably coincide with the axial right line passing through the centres of gravity of all cross-sections of the member taken at right angles thereto.

6. The principle of symmetry in designing must be carried even into the riveting; and groups of rivets must be made to balance about centre lines and central planes to as great an extent as is practicable.

7. In all structural metal-work, excepting only the machinery for operating movable bridges, no torsion on any

member shall be permitted, if it can possibly be avoided; otherwise, the greatest care must be taken to provide ample strength and rigidity for every portion of the structure affected by such torsion.

8. In designing all pin-connected work ample clearance for packing must be provided, and ample room must be left for assembling members in confined spaces.

9. In bridges, trestles, and elevated railroads the thrust from braked trains and the traction must be carried from the stringers or longitudinal girders to the posts or columns without producing any horizontal bending moment on the cross-girders.

10. In trestles and elevated railroads, the columns must be carried up to the tops of the cross-girders or longitudinal girders, and must be effectively riveted thereto. In no case will it be permitted to cut off the columns and rest the cross-girders or longitudinal girders on top of same.

11. Every column that acts as a beam also must have solid webs at right angles to each other, as no reliance shall be placed on lacing to carry a transverse load down the column.

12. In trestles and elevated railroads, every column must be anchored so firmly to its pedestal that failure by overturning or rupture could not occur in the neighborhood of the foot if the bent were tested to destruction.

13. The amount of field-riveting must be reduced to a minimum, without, however, diminishing the number of rivets requisite for strength and rigidity. Whenever it is practicable, all designs are to be made so that the field-rivets can be driven readily.

14. Rivets are not to be used in direct tension.

15. For members of any importance, more than two rivets are to be used for each connection.

16. In designing short members of open-webbed, riveted work, it is better to increase the sectional area of the piece from ten (10) to twenty-five (25) per cent beyond the theoretical requirement than to try to develop the strength by using supplementary angles at the ends to connect to the plates.

17. Star struts formed of two angles with occasional short

pieces of angle or plate for staying same are not to be used, for better results are obtained by placing the angles in the form of a T.

18. In all main members having an excess of section above that called for by the greatest combination of stresses, the entire detailing is to be proportioned to correspond with the utmost working capacity of the member, and not merely for the greatest total stress to which it may be subjected. In this connection, though, the reduced capacity of single angles connected by one leg only must not be forgotten.

19. Designs must invariably be made so that all metal-work after erection shall be accessible to the paint-brush, excepting, of course, those surfaces which are in contact with each other or with the masonry. This requirement rules out all closed columns of every type and description.

20. In general, details must always be proportioned to resist every direct and indirect stress that may ever come upon them under any probable circumstances, without subjecting any portion of their material to a stress greater than the legitimate corresponding working-stress.

21. In all designs simplicity in both main members and details is to be considered of the greatest importance.

22. In all structures rigidity is to be considered quite as important an element as mere strength.

23. Structures on skews are to be avoided whenever it is practicable to do so.

24. The use of more than a single system of cancellation in bridges shall be confined entirely to lateral systems and sway-bracing, except that at mid-panels of trusses two rigid diagonals connected at their intersection may for appearance be employed, provided that either diagonal have sufficient strength to carry the entire shear in either tension or compression, and that the adjacent vertical posts be figured accordingly.

25. The use of redundant members in structures shall not be allowed, excepting only in the case just mentioned of rigid mid-panel diagonals.

26. In all designing true economy must be given the utmost

consideration, and no useless material must be employed, every pound of metal in the structure having a legitimate function; but economy of material must not be quoted as an excuse for using inferior details or scamping the work in respect to strength, rigidity, or appearance.

27. In all structural work the subject of æsthetics must be duly considered; and all designs are to be made in harmony with the principles thereof, to as great an extent as the money available for the work will permit or as the environment of the structure calls for.

RIVETING.

The rivets used shall generally be seven eighths ($\frac{7}{8}$) inch in diameter, smaller ones being employed for small channel flanges and legs of angle-irons less than three and a half ($3\frac{1}{2}$) inches wide. In very heavy work the rivet diameter should be increased to fifteen sixteenths ($\frac{15}{16}$) inch, and in certain extreme cases to one inch.

The least diameters for rivets in flanges of channels are as follows, and the greatest diameters must not exceed the same by more than one sixteenth ($\frac{1}{16}$) of an inch:

Depth of Channel....	6″	7″	8″	9″	10″	12″	15″
Diameter of Rivet....	5/8″	5/8″	3/4″	3/4″	3/4″	3/4″	7/8″

The pitch of rivets in all classes of work in the direction of the stress shall never exceed six (6) inches, or sixteen (16) times the thickness of the thinnest outside plate, nor be less than three (3) diameters of the rivet. At the ends of compression-members it shall not exceed four (4) times the diameter of the rivets, for a length equal to twice the width of the member.

When two or more thicknesses of plate are riveted together in compression-members, the outer row of rivets shall not be more than four (4) diameters from the side edge of the plate.

No rivet-hole centre shall be less than one and a half ($1\frac{1}{2}$)

diameters from the edge of a plate, and, whenever practicable, this distance is to be increased to two (2) diameters.

The rivets when driven must completely fill the holes.

The rivet-heads must in general be round; and they must be of uniform size for the same-sized rivets throughout the work. They must be neatly made and concentric with the rivet-holes, and must thoroughly pinch the connected pieces together.

Rivets with flat heads shall be preferred to countersunk rivets; the height or thickness of the flat head shall be three eighths ($\frac{3}{8}$) of an inch.

Rivets shall not be countersunk in plates less than seven sixteenths ($\frac{7}{16}$) of an inch in thickness.

Flanges of stringers and girders carrying the vertical load from the ties shall have their rivets spaced uniformly from end to end, and at the minimum distance employed.

Whenever possible, all rivets shall be machine-driven, and the machines must be capable of retaining the applied pressure until after the upsetting is completed.

Field-riveting must be done with a button sett: the heads of the rivets must be hemispherical, and no rough edges must be left.

All rivets in splice or tension joints are to be arranged symmetrically so that each half of any tension-member or splice-plate shall have the same uncut area on each side of its centre line.

No rivet, excepting those in shoe-plates and roller or bed plates, is to have a less diameter than the thickness of the thickest plate through which it passes.

The effective diameter of any rivet shall be assumed the same as its diameter before driving; but, in making deductions for rivet-holes in tension-members, the diameter of the holes shall be assumed one eighth ($\frac{1}{8}$) of an inch larger than that of the rivet. In the effective area of riveted members, pin, bolt, and rivet holes shall be counted out for tension, and bolt and pin holes shall be counted out for compression.

DETAILS OF DESIGN FOR ROLLED I-BEAM SPANS.

Rolled I beams used as longitudinal girders shall have preferably a depth not less than one twelfth ($\frac{1}{12}$) of the span. They shall be proportioned by their moments of inertia.

I beam spans may have either one or two beams per rail. In the former case the spacing should be six (6) feet six (6) inches, and in the latter case two (2) feet six (6) inches between contiguous girders. With two lines of stringers per track, there will be required a bracing-frame at each end of span and diagonal bracing between the top flanges, unless the span be less than ten (10) feet in length, in which case the diagonals may be omitted.

With four lines of stringers per track, no diagonal bracing will be required, but three (3) bracing-frames at each end will be used, with three (3) more at mid-span when the span length exceeds ten (10) feet.

Each I beam is to have at each end a pair of stiffening angles, one of which will form a portion of the end bracing-frame. These are to fit tightly at both top and bottom against the flanges.

Under each end of each I beam there is to be riveted a bearing plate of proper area and thickness to distribute the load uniformly over the masonry, said plate being bolted effectively to the latter with due provision for expansion and contraction.

DETAILS OF DESIGN FOR PLATE-GIRDER SPANS.

Plate girders shall have preferably a depth not less than one tenth ($\frac{1}{10}$) of the span.

All plate girders, whenever it is practicable, shall be built without splices in the web; and, when such become necessary, the smallest possible number of same shall be adopted. The splice-plates and rivets for the splices shall be such as to develop in every respect the full strength of the net section of the web, the main splice-plates extending from flange to flange and having at least two (2) rows of rivets on each side of the joint. In addition to these, each flange shall be spliced by two cover-plates on top of the vertical legs of the flange angles. These must be long enough to develop by the connecting rivets at least twenty-five (25) per cent more than the full strength of their net section.

Splices in flange-plates and angles must always be avoided when sufficiently long plates and angles are procurable, which will always be the case, unless the span be abnormally long. Where flange-splices are unavoidable, they must be so located that no two pieces of either the flange or the web shall be spliced within two (2) feet of each other, and so that no flange-splice shall occur at any point where there is not an excess of sectional area above the theoretical requirements. Every non-continuous flange-piece shall be fully spliced so that the splicing plates and rivets shall have a calculated strength at least twenty-five (25) per cent greater than that of the section spliced. Field-splicing of plate girders will never be allowed for fixed spans, except in structures for foreign countries.

At least one half of every flange section must consist of angles, or else the heaviest sections of the latter must be used; and the number of cover-plates must be made as small as practicable, in no case exceeding three (3) per flange. The lengths of these cover-plates must be such as to make them project at each end not less than nine (9) inches beyond the point determined by the calculations for the requisite resistance to bending.

Where two or three cover-plates per flange are used, they shall be of equal thickness, or shall decrease in thickness outward from the angles. The cover-plates shall not extend more than four (4) inches or eight (8) times the thickness of the outer plate beyond the outer line of rivets. With cover-plates more than fourteen (14) inches wide, four (4) lines of rivets shall be used.

The compression-flanges of plate girders shall be made of the same gross section as the tension-flanges; and they shall be so stiffened laterally that the unsupported length shall never exceed twelve (12) times the width of flange.

In deck-spans there are to be bracing frames at the ends and at intermediate points not more than fifteen (15) feet apart; and there is to be an effective system of diagonal bracing of angles between the top flanges of the contiguous girders for each track.

In half-through spans the girders are to be divided up into panels generally not exceeding fifteen (15) feet in length. If a steel floor system be used, there are to be brackets of web-plates and angles at the ends of the cross-girders extending to the top flanges of the longitudinal girders, so as to stay the latter effectively; while, if a wooden floor system of ties resting on shelves or on the bottom flanges be used, there are to be steel cross frames with solid webs, of the greatest depth obtainable, with similar brackets at their ends for the same purpose. Half-through plate-girder spans are to have a rigid, double-intersection, lower lateral system of angles riveted together by plates and angles at their intersections and to the bottom flanges of the steel stringers, if the latter be employed.

Web-stiffeners shall be placed at the ends of plate-girder spans, also at all points of concentrated loading and at intermediate points at distances not exceeding either the depth of the girder or five (5) feet, except in the case of shallow girders where the shear, including impact, does not exceed five thousand (5000) pounds per square inch of web section. Under such circumstances the spacing of intermediate stiffeners may be made as great as three (3) feet six (6) inches.

All stiffeners must bear tightly at top and bottom against the flange angles. Under end stiffeners there must be fillers flush with the flange angles, but intermediate stiffeners shall, preferably, be crimped.

End stiffening angles shall in no case be less than $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$, net, and must have sufficient area to carry the entire end shear, including impact, with the specified intensity of working-stress, no reliance being placed on the fillers.

The sections of intermediate stiffening angles shall not be less than those given in the following table.

Length of Girder.	Dimensions of Angles.
Up to 50′	$3\frac{1}{2}' \times 3\frac{1}{2}'' \times \frac{3}{8}''$
From 50′ to 70′	$4 \times 3\frac{1}{2} \times \frac{3}{8}$
From 70′ to 90′	$5 \times 3\frac{1}{2} \times \frac{3}{8}$

In proportioning the flanges of plate girders, one eighth ($\frac{1}{8}$) of the gross area of the web is to be assumed as concentrated at the centre of gravity of each flange; or, in other words, after having found the net sectional area required for the tension-flange by ignoring the resistance of the web to bending, there is to be subtracted therefrom one eighth ($\frac{1}{8}$) of the gross area of the web-plate.

At the ends of all plate girders there must be sufficient rivets in each flange to transfer properly thereto from the web the total end-shear in a distance equal to the effective depth of the girder.

At the ends of cover-plates the spacing of the rivets which attach the covers, for a length equal to at least twice the width thereof, shall be made the minimum used in the flanges.

Under each end of each plate girder there is to be riveted a bearing plate of proper area and thickness and thoroughly stiffened so as to distribute the load uniformly over the masonry, said plate being bolted effectively to the latter with due provision for expansion and contraction.

DETAILS OF DESIGN FOR OPEN-WEBBED, RIVETED GIRDER-SPANS.

All open-webbed, riveted girders for both deck and half-through bridges shall be riveted up completely in the shop, as field-riveting will be allowed only for the lateral bracing, except in structures for foreign countries.

In open-webbed, through, riveted girders, however, the connection of main members will have to be by field-rivets. In such cases all of the truss-members will have to be assembled in the shop, after which the rivet-holes for the connections shall be reamed so as to ensure perfect fitting in the field.

The use of shallow open-webbed, riveted girders shall be avoided whenever possible, for the reason that they are quite as expensive and never as satisfactory as plate girders. In case, though, of their being required, as for instance in elevated railroads occupying city streets, they are to be provided with short, substantial web-plates at the ends and at all intermediate points where connections are made to other girders.

In proportioning the web-members of such girders, the specified intensities of working stresses are to be reduced from ten (10) per cent for $6'' \times 3\frac{1}{2}''$ angles to twenty-five (25) per cent for equal-legged angles, with proportionate amounts for angles of intermediate inequality of legs, so as to compensate for the secondary stresses due to the eccentric grip of the rivets. In no case will it be permissible to use flats instead of angles for web-members, but tees may be employed, provided their heads be wide enough to permit of satisfactory riveted connections.

At all intersections of web-members with chords, connecting plates are to be used; for it is not permissible to attach web angles directly to chord angles without using an intermediary plate.

The exact intersection at a point of all gravity lines of girder-members assembling at any apex must be adhered to in the designing of open-webbed, riveted girders.

In designing all riveted connections, the greatest care is to be taken to make connecting plates and groups of rivets balance about centre lines of stress, especially where passing from riveted work to pin-connected, as in the case of a riveted span with hinged ends at pedestals.

In all other particulars, the designing of open-webbed, riveted work is to comply, wherever practicable and proper, with the specifications for plate-girder and pin-connected spans.

DETAILS OF DESIGNS FOR PIN-CONNECTED SPANS.

The sections of the top chords and those of the inclined end posts of through-spans shall consist generally of two built channels and a cover-plate, each channel being formed of a web and two angles, the upper one small and the lower one much larger, so as to bring the centre of gravity of the entire box section of the member as close as possible to the mid-plane of the web-plates. In no case will more than one cover-plate be allowed, and this is to be made as thin as is proper. It is permissible to substitute rolled channels for the built ones; but when this is done it is often advisable to rivet a thick narrow plate to the under side of each channel, in order to facilitate the packing and detailing of web-members by keeping the centre line of stress coincident with the gravity axis of the piece.

Main vertical posts shall, generally, be composed of two laced channels, preferably rolled ones, although built ones can be used where large sections are required.

Secondary vertical posts may be built of two rolled channels laced, or of four angles in the form of an I with a single line of lacing. These secondary vertical posts should, preferably, be riveted to the top chord instead of being pin-connected like the main vertical posts.

The channels of vertical posts may have their flanges turned either inward or outward as desired, or so as to best suit the general detailing of the truss.

Stiff bottom chords and inclined web-struts may be made of either two channels with two lines of lacing or of four angles with one line of lacing.

Upper lateral struts, overhead transverse struts, and web-stiffening struts shall, preferably, be made of four angles with one line of lacing. In case, however, the said angles be spaced very far apart, as in lateral struts connecting deep top chords, they are to be placed on the corners of a rectangle, with their legs turned inward, and laced on all four faces of the box strut thus formed.

Eye-bars are to be used for all bottom chords and main diagonals that do not require to be stiffened.

Counters, when employed, can be of either rounds, squares, or flats. These and all other adjustable members are to have their ends enlarged for the screw-threads (unless soft-steel, cold-pressed threads be used) so that the diameter at the bottom of the thread shall be one eighth ($\frac{1}{8}$) of an inch greater than that of the body of a round rod of area equal to that of the adjustable piece.

In short spans, two angles riveted back to back, or even a single large angle, may be used for lower lateral diagonals; but for long spans the diagonals are to be made of four angles in the form of an I with a single line of lacing. When two angles are used, a single plate must not be depended on to form the splice at the intersection of the diagonals, but two angles, each not less than two (2) feet long, are to be placed beneath or on top of the spliced angles, so as to form a full splice in respect to rigidity as well as strength.

Diagonals for upper lateral systems and vertical sway-bracing shall, preferably, be built of four angles in the form of an I with a single line of lacing; but, for structures where this section would involve an extravagant use of metal, two of the angles, one at top and one at bottom, may be omitted, thus making each strut consist of two angles laced, provided, of course, that where the struts cross they shall be rigidly connected by two plates of ample size. This unbalanced section for such diagonals is to be avoided whenever it can be done without undue use of metal. In no case, though, will it be permissible to use angles in tension that are not capable of resisting properly the possible compressive stresses, with due regard for the specified limit of ratio of unsupported length to least radius of gyration.

In designing transverse lateral and overhead struts and their connections it must be remembered that their main function is to hold rigidly the chords or posts to place and line, and not merely to resist as columns the greatest calculated direct stresses to which they may be subjected. For this reason such struts should have ample section for rigidity, and the connecting plates at their ends should grip both connected members effectively.

Stringers for truss-bridges shall invariably be built of plates and angles, and no cover-plates will be allowed for the flanges. Their depths shall be made not less than the most economic ones in respect to weight of metal required, provided that the bridge clearance will permit, and never less than one twelfth ($\frac{1}{12}$) of the span. No splices will be allowed in their flanges nor any in their webs, provided that sufficiently long web-plates are procurable. The compression-flanges shall be made of the same gross section as the tension-flanges; and they shall be so stiffened that the unsupported length shall never exceed twelve (12) times the width of flange.

Rigid diagonal bracing of angles is invariably to be used between the top flanges of stringers, and rigid bracing-frames are to be employed near all expansion points. If the panel length exceed thirty (30) feet, there shall be a bracing-frame at mid-length between the contiguous stringers of each track; but for all shorter panels the rigid lower lateral diagonals which are riveted to the bottom flanges will stiffen the latter sufficiently.

In respect to stiffening angles for stringers, the rules governing those for plate-girder spans are to be followed; but the end stiffeners are to be faced or otherwise treated so as to make the stringers of exact length throughout, and so as to effect a uniform bearing of the end stiffeners against the webs of the cross-girders.

In respect to proportioning of flanges and number of rivets required, the rules given for plate-girder spans are to apply also to stringers. The said rules are to apply also to cross-girders, as shall also those relating to stiffeners, splices, cover-plates, and size of compression-flanges, that are given for plate-girder spans. Wherever it is necessary to notch out the corners of the cross-girders to clear the chords, the greatest care must be taken to provide an adequate means for transferring the shear to the posts without impairing either the strength or the rigidity. If necessary, in through-bridges the web of the cross-girder can be divided into three parts so as to let the end portions project above the top flange and form brackets that will afford opportunity for using an ample number of rivets to connect to the posts, and will strengthen properly the otherwise weakened cross-girder.

In order to carry the thrust of trains from the stringers to the posts through the lower lateral diagonals, the latter and the stringers are to be made to form complete horizontal trusses by running angles between stringers at the level of the bottom flanges. In single-track bridges two pieces of angles per panel running transversely between stringers at the intersection of the latter with the diagonals will suffice; but in double-track bridges there will be required two such angles per panel between inner stringers, and four diagonal angles per panel to run from where the lateral diagonals intersect the outer stringers to where the inner stringers meet the cross-girders.

All plates, angles, and channels used in built members of trusses must, if practicable, be ordered the full length of the member; otherwise the splices must develop the full strength of the member, without any reliance being placed on the abutting ends for carrying compression.

But in total splices at the ends of sections perfect abutting of the dressed ends is to be relied upon. However, the splice-plates even there must be of ample size and strength for both rigidity and continuity.

The unsupported width of plates strained in compression, measuring between centre lines of rivets, shall not exceed thirty-two (32) times their thickness, except in the case of cover-plates for top chords and inclined end posts, where the limit may be increased to forty (40) times the thickness. Where webs are built of two or more thicknesses of plate, the rivets that are used solely for making the several thicknesses act as one plate shall in no case be spaced more than twelve (12) inches from each other or from other rivets connecting said component thicknesses together. The least allowable thickness for such compound web-plates shall be one (1) inch.

The open sides of all compression-members composed of two rolled or built channels, with or without a cover-plate, shall be stayed by tie plates at ends and by diagonal lacing-bars or lacing-angles at intermediate points. Lacing-bars may be connected to the flanges by either one or two rivets at each end; but lacing-angles, which are used for members of heavy section only, must be connected by two rivets at each end.

The tie-plates shall be placed as close as practicable to the ends of the compression-members. Their thickness shall not be less than one-fiftieth ($\frac{1}{50}$) of the distance between the centre lines of the rivets by which they are connected to the flanges, unless said tie-plates be well stiffened by angles, in which case they may be made as thin as three eighths ($\frac{3}{8}$) of an inch.

The length of a tie-plate shall never be less than its width, or one and one-half ($1\frac{1}{2}$) times the least dimension of strut (unless it be close to a web diaphragm of the member, in which case it may be as short as twelve (12) inches), and seldom greater than one and one-half ($1\frac{1}{2}$) times its width.

The thicknesses of lacing-bars shall never be less than one fiftieth ($\frac{1}{50}$) of the length between centres of the end rivets, measuring between inmost rivets in case that there be more than one rivet at each end. The smallest section for a lacing-bar shall be one and three quarter ($1\frac{3}{4}$) inches by three eighths ($\frac{3}{8}$) of an inch, which size shall be used for channels under nine (9) inches deep; and the largest section shall be two and a half ($2\frac{1}{2}$) inches by one-half ($\frac{1}{2}$) inch, which size shall be used for channels fifteen (15) inches deep. For intermediate sizes of channels, the sizes of lacing-bars shall be interpolated. For all built channels of greater depth than fifteen (15) inches, and for all cases where a lacing-bar would require a greater thickness than one-half ($\frac{1}{2}$) inch, angle lacing is to be used, the smallest section for same being $2'' \times 2\frac{1}{2}'' \times \frac{3}{8}''$, and the largest $2\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$. For two (2) inch lacing-bars and two and a half ($2\frac{1}{2}$) inch lacing angles, three-quarter ($\frac{3}{4}$) inch rivets are to be used; and for two and a half ($2\frac{1}{2}$) inch lacing-bars and three (3) inch lacing-angles seven-eighths ($\frac{7}{8}$) inch rivets are to be adopted.

In general, the inclination of lacing-bars to axis of member shall be about sixty (60) degrees; but in members of minor importance and in tension-members the said inclination may be made slightly flatter.

Pin-plates shall be used at all pinholes in built members for the double purpose of reinforcing for the metal cut away and reducing the intensity of pressure on pin and bearing to or below the specified limit. They shall be of such size as to distribute properly, through the rivets, the pressure carried by such plates to both flanges and web of each segment of the member; and they shall extend at least six (6) inches within the tie-plates of said member, so as to provide for not less than two (2) transverse rows of rivets there.

When the pin ends of compression-members are cut away into jaw-plates or forked ends, for the purpose of packing closely the various members connected by the pin, these jaw-plates or post extensions shall be considered as columns, the thickness of each of which shall be determined by the following formula:

$$p = 10,000 - 300\frac{l}{t};$$

where p is the greatest allowable intensity of working-stress (impact being considered); l is the unsupported length in inches, measuring from the centre of the pinhole to the centre of the first transverse line of rivets beyond the point at which the full section of the member begins; and t is the total thickness in inches of one jaw. The length l is always to be made as small as practicable; and, in cases of unavoidably long extensions, the plates are to be stiffened by an interior diaphragm composed of a web with four, or sometimes only two, angles.

It is always better, whenever practicable, to avoid cutting away the ends of channels; but, if they must be trimmed, the ends must be reinforced so that the strength of the member shall not be reduced by the trimming.

In riveted tension-members, the net section through any pinhole shall have an area fifty (50) per cent in excess of the net sectional area of the body of the member. The net section outside of the pinhole along the centre line of stress shall be at least sixty-five (65) per cent of the net section through the pinhole.

Pins are to be proportioned to resist the greatest shearing and bending produced in them by the bars or struts which they connect. No pin is to have a diameter less than eight tenths ($\frac{8}{10}$) of the depth of the deepest eye-bar coupled thereon. No truss-pin is to have a smaller diameter than three and a half ($3\frac{1}{2}$) inches, and no lateral pin, if any such be used, a diameter less than two and a half ($2\frac{1}{2}$) inches.

Lower chords are to be packed as closely as possible, and in such a manner as to produce the least bending moments on the pins; but adjacent eye-bars in the same panel must never have less than a one half ($\frac{1}{2}$) inch space between them, in order to facilitate painting. The various members attached to any pin must be packed as closely as practicable, and all interior vacant spaces must be filled with steel fillers, where their omission would permit of motion of any member on the pin. All bars are to lie in planes as nearly as possible parallel to the central truss-plane, no divergence exceeding one eighth ($\frac{1}{8}$) of an inch to the foot being permitted.

In detailing I struts composed of four angles with a single line of lacing, the clear distance between backs of angles shall never be made less than three-quarters ($\frac{3}{4}$) of an inch, in order to permit the insertion of a small paint-brush.

The greatest allowable pressure upon expansion-rollers of fixed spans, when impact is considered, shall be determined by the equation

$$p = 600d,$$

where p is the permissible pressure in pounds per lineal inch of roller, and d is the diameter of the latter in inches. The least allowable diameter for expansion-rollers is three (3) inches.

Rollers shall be enclosed in boxes made practically dust-tight, but which will not retain water, and which are so designed that the sides can be readily removed for the purpose of cleaning. These boxes must be so designed as to permit of a free movement of the rollers in the longitudinal direction of span sufficient to take up the extreme variations in length due to temperature changes and deflection, and at the same time prevent any transverse motion of the end of the span.

All shoe-plates, bed-plates, and roller-plates are to be so stiffened that the extreme fibre-stress under bending, when impact is included, shall not exceed sixteen thousand (16,000) pounds.

Pedestals shall be either of cast steel or built up of plates and shapes. In built pedestals, all bearing surfaces of the base-plates and vertical bearing-plates must be planed. The vertical plates must be secured to the base by angles having at least two rows of rivets in the vertical legs; and the said vertical plates must bear properly from end to end upon the base. No base-plate, vertical plate, or connecting angle shall be less in thickness than three quarters (¾) of an inch. The vertical plates shall be of sufficient height and must contain enough metal and rivets to distribute properly the loads over the bearings or rollers. The bases of all cast-steel pedestals shall be planed so as to bear properly on the masonry or rollers. All rollers and the faces of base-plates in contact therewith are to be planed smooth, so as to furnish perfect contact between rollers and plates throughout their entire length.

All pedestals whether built or cast must have one or more diaphragms between webs, carried up as high as the general detailing will permit, so as to transmit transverse horizontal thrust to the base without overstraining the webs by bending in their weakest direction.

Heads of eye-bars are to be made of such dimensions that when the bars are tested to destruction they shall break in the body and not in the eyes; and in case of loop-eyes, so that they shall not fail in the welds. Rods with bent eyes shall not be used. In loop-eyes, the distance from the inner point of the loop to the centre of the pinhole must not be less than two and one half (2½) times the diameter of the pin, and the loop must fit closely to the pin throughout its entire semicircumference.

DETAILS OF DESIGN FOR TRESTLES AND ELEVATED RAILROADS.

The sections of main members of trestles shall, generally, be as follows: Columns, two channels laced with flanges turned either out or in, two channels with I-beam web between, four Z bars with web-plate, four Z bars with a single line of lacing inside and occasional stay-plates outside, or four angles with a single line of lacing inside; diagonals in transverse and longitudinal bracing, and all bottom horizontal bracing struts, four angles with a single line of lacing; horizontal transverse bracing struts at top of towers, bracing frames of angles; longitudinal struts at top of towers, plate girders; and longitudinal girders, plate-girder spans, or occasionally, for very long spans, open-webbed, riveted girders or pin-connected trusses.

The detailing for longitudinal girders of trestles and elevated railroads and the bracing between same shall comply with the specifications governing the designing of plate-girder spans and the floor systems of pin-connected spans.

In general, the transverse and longitudinal bracing of trestle towers shall consist of a double-cancellation system of stiff diagonals without any horizontal struts, except at the bottom between pedestals. The latter struts must be strong enough to move the column feet upon their sliding-bearings when said struts are expanded or contracted by changes of temperature. Provision must be made for holding some feet rigidly, and for sliding some in one horizontal direction only, and others in any horizontal direction, at the same time holding them all down so that they shall not be lifted perceptibly by the wind-pressure. Sliding-plates are always preferable to rollers for pedestals of trestles. They shall be planed extremely smooth, and so as to bear properly at all parts.

Occasionally, in solitary bents, it is permissible to use hinged ends for columns at pedestals; but it is generally better to make them fixed, and to figure the columns for the greatest bending produced in them by transverse loads and extreme changes of temperature.

The tops of trestle columns are to be made vertical by bending them just beneath the longitudinal girders where the latter are riveted to them; and the upper transverse struts must be made as deep as the longitudinal girders, and must be riveted effectively to the columns. Corner brackets of double webs are to be used for connecting the columns to the horizontal struts and bracing-diagonals, and at the same time to strengthen the column at the bend. Additional strengthening is to be given by using a solid web or diaphragm in the column, extending from the top thereof to a point about two (2) feet below the bend.

All splices in columns are to be full, butt splices, located preferably about two (2) feet above the points where the sway-diagonals connect, shingle-splicing being avoided because of the trouble it gives during erection.

The best span lengths for trestles are generally those which make the total cost of structure a minimum, the tower length varying from twenty (20) feet for low trestles to thirty (30) feet for very high ones, and the intermediate spans varying from thirty (30) to sixty (60) feet for the same limiting heights. Any length of girder exceeding sixty (60) feet would probably necessitate the employment of a too long, heavy, and expensive traveller, or else the use of bents of falsework between the towers.

For elevated railroads the sections of main members shall be as follows: Longitudinal girders, preferably plate girders, or, if necessary, open-webbed, riveted girders; cross-girders, plate girders; columns for structures without longitudinal or tower bracing, two channels with an I beam riveted between; and columns for structures with longitudinal or tower bracing, four Z bars with a web-plate.

All columns for elevated railroads are to have both ends fixed, being held rigidly at the top by either the longitudinal girders or by deep struts that carry the thrust of braked trains from the track to the columns, and their sectional areas are to be figured accordingly for both direct load and bending.

Longitudinal girders in elevated railroads shall, generally, be riveted into the cross-girders and not rest thereon, except under certain conditions for the sake of clearance beneath, in which case the top flanges of the half-through girders must be stayed at the ends and at intermediate points, as specified for plate-girder spans

On all curves in elevated railroads, special lateral bracing of angles, riveted at intersections to the longitudinal girders and carried over and riveted to the columns, must be employed.

Where brackets for columns can be used advantageously in elevated-railroad work, they must be put in, and must be built of solid web plates and angles.

In general, the limiting length of structure between expansion points shall be about one hundred and fifty (150) feet. If this length be exceeded materially, the columns may have to be strengthened to resist the bending caused by changes in temperature.

All expansion-pockets are to be so detailed as to throw the load from the longitudinal girder as close as possible to the web of the cross-girder; and sufficient rivets are to be used in connecting the pocket to the cross-girder to provide for both the direct shear and the bending moment from the eccentric load.

All anchor-bolts at column feet are to extend well up above the base-plate, passing inside of a curved plate that is riveted to the column, and which supports a heavy washer-plate to receive the anchor-bolt nut. The space between the curved plate and the anchor-bolt after erection is to be filled with Portland-cement grouting.

All column feet are to be raised so far above the ground that no dirt, snow, or moisture can collect around them and remain there. The boxed spaces at column feet are to be filled with Portland-cement concrete made with small broken stone.

The bases of pedestals are always to be made large enough to prevent all possibility of settlement of foundations. In figuring the pressure on the base of the pedestals it is not sufficient to recognize only the direct live and dead loads, but it is necessary also to compute the additional unequal intensities of loading caused by both longitudinal and transverse thrusts.

CHAPTER XV.

SPECIFICATIONS FOR RAILROAD DRAW-SPANS.

THE specifications given in the preceding chapter for fixed spans apply also to draw-spans, except where otherwise stated in the following pages.

GENERAL DESCRIPTION.

MATERIALS.

The specifications previously given apply also to draw-spans, except that cast iron may be used for the centre castings on top of pivot-piers, for anchor-pieces in the masonry, for shafting boxes, and for rail-chairs, and some other castings of minor importance. The use of high steel for drawbridges will not be permitted.

STYLES OF BRIDGES FOR VARIOUS SPAN LENGTHS.

For spans up to one hundred and sixty (160) feet in length, plate-girder spans should be used. These may be made to act as continuous girders over the pivot-pier, or may have pin-connections over the drum, so that when the live-load is applied they will act as two separate spans. The latter style is generally preferable, because there is no tendency for the far end of the span to rise when the live load is being brought on.

For spans between one hundred and sixty (160) feet and two hundred and seventy-five (275) feet, pin-connected Pratt trusses with parallel top chords and stiff diagonals in panels where there is reversion of stress, or riveted trusses of single cancellation, are to be used.

For spans between two hundred and seventy-five (275) feet and three hundred and fifty (350) feet, pin-connected Pratt trusses with broken top chords are to be employed.

For spans of over three hundred and fifty (350) feet, pin-connected trusses with subdivided panels are to be adopted.

It is understood that these limiting lengths are not fixed absolutely, as the best limits will vary somewhat with the number of tracks and the weight of trains.

The height of towers should generally be between one sixth ($\frac{1}{6}$) and one seventh ($\frac{1}{7}$) of the total length of span, measuring from centre to centre of end-pins; although in certain cases it may, for the sake of appearance, be made a little greater. The truss depth at the inner hips should be from one ninth ($\frac{1}{9}$) to one tenth ($\frac{1}{10}$) of the total length of span. The truss depth at outer hips for spans up to four hundred (400) feet will generally be determined by the clearance required. For longer spans it should be between one fourteenth ($\frac{1}{14}$) and one fifteenth ($\frac{1}{15}$) of the total span length.

The length of the centre panel will, in most cases, be made equal to the perpendicular distance between central planes of trusses.

In spans having horizontal top chords all panels of the latter must be made of stiff members, excepting only the centre panel over the pivot-pier; and the diagonals next to the middle panel are to be tension-members.

Broken top chords must be made of stiff members from ends to inner hips, but the portion between the inner hips is to be made of eye-bars. Inclined posts extending from inner hips to drum are to be used in all cases where top chords are broken.

LOADS.

The loads to be considered in designing draw-spans are the following:

A. Live Load.
B. Impact Allowance Load.
C. Dead Load.
D. Uplift at Ends.
E. Direct Wind Load.
F. Indirect Wind Load or Transferred Load.

LIVE LOADS.

The live loads for the various parts of the structure are to be taken from the "Compromise Standard System of Live Loads for Railway Bridges," in the same manner as previously specified for fixed spans.

The live load for trusses with only one arm loaded is to be taken from the live-load curves for a span equal to the distance between the centre of the end-pin and that of the pin at the foot of the nearer tower post; but for both arms loaded the live load is to be taken for a span equal to the distance between centres of end-pins.

For only one arm loaded, the half-span is to be considered to act as a simple span on two supports; and, for both arms loaded, the entire span is to be considered continuous over four supports. The stresses due to the live load, with both arms wholly or partially loaded, are to be determined by the balanced-load method. For convenience in determining the reactions at ends and at centre supports for balanced loads the curve given on Plate IX can be used. This gives the percentage of any balanced load which is supported at the outer end of a half-span.

DEAD LOADS.

In spans over two hundred and seventy-five (275) feet, the dead load per truss is to be increased properly from the ends towards the centre of span in order to cover the weight of the heavy truss-members, which increase in size toward the centre of the span. The division of the dead load between top and bottom chords is to be the same as specified for fixed spans.

The dead loads from tower, drum, and turntable are not to be considered as affecting the stresses in the trusses.

ASSUMED UPLIFT LOADS.

There will be a considerable uplift at the ends of the span, for they are to be brought to a firm bearing by means of the end-lifting device. The amount of this uplift per truss or girder is to be assumed as a certain proportion of the entire dead load carried by one arm of the said truss or girder when the span is being swung, which proportion is to be taken from the following table:

Spans.	Ratios of Uplift to Dead Load.
Up to 150′	$\frac{1}{3}$
150′ to 250′	$\frac{1}{4}$
250′ to 350′	$\frac{1}{5}$
350′ to 450′	$\frac{1}{6}$
Over 450′	$\frac{1}{7}$

These uplifts are to be adopted both for finding the uplift stresses in trusses and for proportioning the end-lifting machinery; provided, however, that for the latter purpose no assumed uplift be less than twenty thousand (20,000) pounds for single-track drawbridges or less than forty thousand (40,000) pounds for double-track drawbridges.

WIND LOADS.

The wind loads per lineal foot of span for both the loaded and the unloaded chords are to be the same as those specified for fixed spans, the length of span, however, being that of one arm of the draw.

When the span is open, all the wind load is to be carried to the drum through the lateral systems. When the draw is closed, the wind load is to be carried to both the ends and the centre supports, the lower lateral system and bottom chords being considered to act as a continuous girder over four supports. The reactions at the ends and the centre can be taken from the curve for balanced live loads.

INDIRECT WIND LOAD OR TRANSFERRED LOAD.

The wind load on the upper chords is to be assumed to travel through the upper lateral system to the inner hips, when the span is open, then down the inner inclined posts to the drum, thus producing a transferred load on the leeward inclined post and a released load on the windward one. As the upper lateral system is not continuous between the inner hips, none of the wind load on the upper lateral system is carried down the tower-posts, excepting that which comes on the centre panel and the two adjacent panels. In order to ensure such a distribution of the wind load it is necessary to put no diagonals in those panels of the upper lateral system which are adjacent to the inner hips and between same and the tower.

When the draw is closed, one half of the wind load on the upper lateral system of one arm is to be assumed to travel down the end inclined posts, and one half down the inner inclined posts.

The transferred-load stress on an inclined post is to be found by multiplying the wind load going to it by the average height of the top-chord panel points to which said wind load is applied, dividing the product by the perpendicular distance between central planes of trusses, and multiplying the quotient by the secant of the angle that the inclined post makes with the vertical.

The transferred-load stress on a tower-post is to be determined by multiplying the wind loads carried by the two opposite posts by the respective heights at which these loads are applied, and dividing the sum of these products by the perpendicular distance between central planes of trusses.

COMBINATIONS OF STRESSES.

In ascertaining the stresses in the trusses of swing-bridges the following conditions are to be considered:

Case No. 1. Greatest stresses, dead load only acting, bridge swinging open.

Case No. 2. Greatest stresses from assumed uplift at end of span.

Case No. 3. Greatest stresses from live load on one arm only; each arm being considered to act as a simple span on two supports.

Case No. 4. Greatest stresses from live load on both arms, the live load advancing from both ends toward the centre until the span is fully loaded; the latter being considered to act as a continuous girder over four supports.

Case No. 5. Greatest direct stresses, on the chords that carry the live load, from wind load when the bridge is open.

Case No. 6. Greatest direct stresses, on the chords that carry the live load, from wind load when the bridge is closed and wholly or partially loaded.

Case No. 7. Greatest indirect wind-load stresses or transferred-load stresses on the lower chords when the bridge is closed and wholly or partially loaded.

The first combination of these stresses includes Cases No. 1, No. 2, No. 3, and No. 4, and gives the greatest stresses for all truss members from combined live and dead loads, for which combination the regular specified intensities of working-stresses are to be used. It is to be noted that wherever the load for Case No. 2 increases the total stress on any member, its effect is to be considered; but wherever the said load decreases the total stress on any member, its effect is to be ignored. The reason for this is that the amount of uplift is a purely arbitrary assumption, which possibly may never be realized. This method of treating the uplift-load stresses causes errors on the side of safety, which do not add materially to the total weight of metal in the structure, and which tend to strengthen the lighter members of the trusses.

The second combination of these stresses includes all seven cases, but it is to be noticed that the only truss members affected by the wind loads are the inclined posts at ends and over drum, and the chords which carry the live load. In this second combination it must not be forgotten that the metal is to be strained thirty (30) per cent higher than in the first combination.

For the lateral systems the following conditions are to be considered:

For upper lateral systems of through-bridges and lower lateral systems of deck-bridges—

Case No. 1. Greatest wind-load stresses when span is swinging.

Case No. 2. Greatest wind-load stresses when span is closed and ends are raised, thus making the entire lower lateral system with the bottom chords a continuous girder with four points of support. This case does not involve the presence of any live load on the span.

For lower lateral systems of through-bridges and upper lateral systems of deck-bridges—

Case No. 3. Greatest wind-load stresses when span is swinging.

Case No. 4. Greatest wind-load stresses when span is closed and ends are raised, and with live load on one arm only, thus making the loaded chords with their lateral system a simple span with supported ends.

Case No. 5. Greatest wind-load stresses when span is closed and ends are raised, and with the live load on both arms covering same either wholly or partially, thus making the loaded chords with their lateral system a continuous girder with four (4) points of support.

The greatest stress on any lateral member found by these five conditions of wind-loading is to be used in proportioning its section, and there is to be assumed no division of the wind load between structure and train, although the failure to make said division will cause small errors on the side of safety.

DETAILS OF DESIGN FOR PLATE-GIRDER DRAW-SPANS.

Plate-girder drawbridges are to be divided into two types, viz.:

Type No. 1. Continuous girders, in which the girders act as continuous spans resting on four points of support; and

Type No. 2. Non-continuous girders, in which the two arms carry the live load independently of each other, the dead-load stresses over the pivot pier when the span is swung being carried by links.

For Type No. 1 the same combinations of stresses are to be used as specified for truss draw-spans, but it will generally be found that the wind loads do not affect the proportioning of the girders

For Type No. 2 the loads to be considered are as follows:

Case No. 1. Dead-load stresses when the span is swung.

Case No. 2. Dead-load stresses for each arm acting independently of the other.

Case No. 3. Live-load stresses for each arm acting independently of the other.

The stresses in Cases No. 2 and No. 3 are to be combined, but those in Case No. 1 are not to be combined with either of the others, the effect of reversion of stress, however, being provided for as specified for fixed spans.

The only effect of wind load to be considered for the girders of Type No. 2 is that upon the connecting links over the turntable when the span is being rotated, for which case the amount of the wind load is to be taken at two hundred (200) pounds per lineal foot of span.

In general, the specifications for the detailing of fixed plate-girder spans are to govern the designing of plate-girder draw-spans, except as hereinafter stated.

In deck, plate-girder draw-spans the girders are to be spaced the same distance apart as specified for fixed plate-girder spans of one half the length. For half-through, plate-girder, draw-spans the girders may be spaced as closely as the previously specified clearance requirements will permit.

For deck-spans four points of support on the drum will suffice, but for half-through spans eight points will be required. The diameter of the drum is to be made as small as practicable, but never less than eight (8) feet; and the distribution of the load over the drum is to be uniform.

All girders are to be thoroughly stiffened at all points of bearing over the drum, and bearing-plates not less than one (1) inch in thickness are to be used between the drum and all girders bearing on same.

For spans of Type 1, when the length over all exceeds ninety (90) or at the utmost one hundred (100) feet, it will be necessary to splice the main girders in the field. These splices must be thoroughly made, shingle or staggered splices only being allowed; and there must be a twenty-five (25) per cent excess of strength in the details at all points thus spliced, as previously specified for fixed plate-girder spans.

Rigid bracing-frames are to be used between main girders of deck-spans at the points where the main girders bear on the drum; and heavy, rig d, plate cross-girders resting on the drum are to be used for half-through spans.

End lifts must be provided for draw-spans of Type No. 1, as hereinafter specified for truss-span drawbridges.

For spans of Type No. 2 the centre panel is to be made with pin connections, the bottom-chord pins resting in pedestals, which furnish proper bearings on the drum. The top-chord tension is to be taken up by eye-bars, which serve as toggles for raising the ends of span. These toggles are to be worked by a screw at centre of span.

The compression in bottom flanges of girders, due to dead load when the span is swung, is to be taken up by struts hinged on the bottom-chord pins.

The eye-bars of the top chords must have slotted eyes, so as to make sure that each half of the girder will act as a simple span when the live load is applied.

Proper shoes must be provided at ends of span, with grooves into which the sole-plates on ends of girders are lowered into place. These grooves should be deep enough to hold the ends of the girders securely, and the toggle at the centre must provide enough lift to clear the ends properly for turning.

All track-rails, guard-rails, and stringers must be discontinuous in the centre panel so that the toggle will be free to act.

The ends of each pair of girders over the drum must be thoroughly braced together.

The end lifting arrangement of these spans demands the most accurate shop-work; and in every case the whole span must be assembled in the shops, so that the lifting machinery can be thoroughly tested before being shipped.

DETAILS OF DESIGN FOR TRUSSES.

The details of trusses for draw-spans shall comply in general with the specifications given for trusses of fixed spans.

In trusses having broken top chords, that portion of said top chords between outer and inner hips is to be made of rigid members, and that portion between the inner hips and over the tower is to be made of eye-bars.

In pin-connected trusses with parallel chords rigid members will be required throughout the top chord, except for the centre panel, in which eye-bars are to be used. In riveted trusses stiff top chords from end to end of span are to be adopted.

The bottom chords are to be of rigid sections throughout for all spans; and for spans over three hundred (300) feet in length provision must be made near the panel points at feet of tower-posts for adjusting, by means of shimming-plates, the height of the ends of the trusses. These shimming-plates must provide an end, vertical adjustment of one (1) inch for each one hundred (100) feet of length of one arm of draw. For spans shorter than three hundred (300) feet shimming-plates beneath the end bearings will give sufficient adjustment.

Rigid portal-bracing must be used between the two inclined posts at both the inner and the outer hips. These portals are to be carried down as low as the specified clearance over tracks will permit.

In heavy spans the portal-bracing must attach to the upper and lower flanges of inclined posts, instead of lying in the gravity-planes of same.

The tower must be rigidly braced in all four faces. In the transverse planes all the diagonals and horizontal struts must, generally, be made of stiff members of box or I sections, so as to take hold of the exterior of the posts; and this sway-bracing must be carried down as low as the specified clearance will permit, so as to hold the tower-posts firmly to place and line.

In the planes of the trusses the diagonals are to be made of adjustable rods of ample section to provide for any possible unequal vertical wind-pressure when the span is open ; and the horizontal struts of box or I sections are to be rigidly attached to the columns by large plates, to which the clevises of the adjustable rods attach by means of pins.

A pair of adjustable diagonal rods or rigid struts must be used in the horizontal plane of each vertical panel of tower-bracing, so as to ensure the permanent rectangularity of the section of the tower.

All splices in top and bottom chords, inclined posts, and tower-posts are to be full splices, so as to develop the full strength of the section, even if the computed stresses do not demand such a strength of detail.

The upper lateral system between the inner and the outer hips is to be made of rigid diagonals, capable of taking both tension and compression, and transverse struts of I section, that take firm hold of the upper and lower flanges of the top chords. From inner hip to inner hip the diagonals are to be of adjustable rods ; but, as before stated, the rods are to be omitted from the panels next to the hips, so as to ensure a proper travel of the wind-loads to the pivot-pier.

The transverse sway-bracing between trusses is to be made entirely of rigid members, and is to be carried down as low as clearance requirements will permit. In long spans the lower horizontal struts of the vertical sway-bracing must take hold of the vertical posts at the flanges of same, so as to hold the said posts firmly in position.

DETAILS OF DRUM AND TURNTABLE.

The drum must be strong enough to distribute the total load from the span properly over the rollers. In general, it should be made, within reasonable limits, as deep as possible, for the cost for the extra depth will be more than offset by the saving in height of pivot-pier.

The bending moment on the drum is to be computed by the compromise formula,

$$M = \frac{l}{10} Wl,$$

where M = bending moment in foot-pounds, W = greatest load in pounds on one point of bearing on drum, and l = distance in feet between points of bearing.

The drum is to be designed according to the specifications for ordinary plate girders. The web thereof shall have stiffeners on both sides at all points of concentration. These stiffeners must have perfect contact with the top and bottom flanges. The section required for these stiffeners is to be determined by considering the entire concentration on one point of bearing to be carried by the said stiffeners, which act as a column, fixed at both ends, with an unsupported length equal to the depth of drum. Stiffeners, each consisting of two angles, placed on opposite sides of the web must be used at intermediate points at distances not exceeding either the depth of web or three (3) feet six (6) inches.

Brackets to support the pinions gearing into the rack are to be provided on the drum. They shall be built of rolled-steel sections, and made amply strong in all directions and in every particular so as to resist the greatest thrust, wrenching, or torsion that can possibly come from the shaft. In no case are these brackets to be made of castings. The use of turned bolts for attaching the brackets to the drum will not be permitted where it is possible to drive rivets, as such bolts do not afford sufficient rigidity to prevent the connections from working loose sooner or later. The splices in the web and flanges of drum must be such as to develop the full strength of same; and the abutting ends of web and flanges must be planed smooth, and have continuous contact.

The drum must be made perfectly round, so that the centre line of web at any height will conform to the circumference of a circle; and, to preserve this form and brace the drum thoroughly, rigid radial struts are to be run from the centre casting to the drum, taking hold of the latter at each point of concentrated loading, and at intermediate points when the bearings are spaced more than eight (8) feet between centres. These radial struts must be made of four angles with solid webs or angle lacing. At the centre they are to be riveted to circular plates fitting closely around the centre casting, thus anchoring the drum firmly to the latter. Oil-grooves must be provided where these plates bear on the centre casting. Fillers are to be used beneath all stiffeners on drum.

The drum must be assembled and the bottom must then be planed smooth so as to provide an even bearing for the upper track. If it is not practicable to plane the entire drum at once, then each segment thereof is to be planed separately; but in this case the greatest care is to be taken to make the assembled parts form a perfect whole.

The least thickness of metal to be used for bottom flanges of drum shall be three quarters ($\frac{3}{4}$) of an inch, so as to provide ample metal for planing off the bottom, and that for the web and top flanges one-half ($\frac{1}{2}$) inch.

The upper track shall be made of segments of sufficient thickness to distribute the load properly between the rollers and the drum. The top face of this track shall be planed smooth so as to form close contact with the bottom flange of the drum, and the lower face shall be planed conical so as to fit closely to the conical rollers. All joints between segments are to be planed smooth and to such bevel as to ensure perfect contact with each other. These track segments are to be riveted or bolted to the bottom flanges of the drum with fifteen-sixteenths ($\frac{15}{16}$) inch rivets or bolts, placed opposite, and spaced not to exceed fifteen (15) inches between centres. The heads of these bolts or rivets are to be countersunk in the track on the side next to the rollers.

No rust-cement or any other composition is to be used between the track and the drum.

The lower track is to made strong enough to distribute the load from the rollers uniformly over the masonry. The bending moment on the lower track is to be found by the formula

$$M = \frac{1}{12} Wl$$

where M = greatest bending on lower track, W = total load on one roller, and l = distance from centre to centre of adjacent rollers, measured on the centre line of the track.

The greatest allowable tensile stress on the extreme fibre for cast-steel track shall not exceed eight thousand (8,000) pounds per square inch, when the effect of impact is included. The lower track shall be made in segments from six (6) to eight (8) feet in length. All abutting ends of lower-track segments are to be planed smooth, are to have close contact throughout, and are to be bolted together by two bolts passing through holes in lugs cast thereon. These bolts are to be at least fifteen sixteenths ($\frac{15}{16}$) of an inch in diameter.

In no case shall the upper track be less than two and one-quarter ($2\frac{1}{4}$) inches, or the lower track less than two and one-half ($2\frac{1}{2}$) inches thick, measuring on the central cylindrical surface of the drum.

The lower track shall be anchored to the top of the pivot-pier with bolts not less than one (1) inch in diameter, nor less than fifteen (15) inches long, set in place with Portland-cement grouting. These bolts are to be made of soft steel, with cold-pressed threads and hexagonal nuts at top, and with split ends and wedges at the bottom. They are to be placed in pairs opposite on the inside and outside of the track, and are to be spaced not to exceed eighteen (18) inches between centres.

The top of the pier is to be levelled off with neat, Portland-cement mortar, and the lower track is to be set in same. It shall be made one and one-half ($1\frac{1}{2}$) or two (2) inches higher in the centre than at the edge, so that the water will drain toward the latter. A small gutter or depression in the top of the pier is to be made just inside of the lower track, and at the bottom of this depression drain-holes are to be put in, leading the water from the gutter down on the outside of the pier. These drain-holes are to be at least two (2) inches in diameter; and the tops are to be protected with screens, so as to prevent choking. They are to be spaced not to exceed ten (10) feet between centres.

The rollers shall be of cast steel, and are to be made solid, excepting only the centre hole and four or more radial holes that are left in the casting for the double purpose of reducing the weight and facilitating a rapid and uniform cooling, the said holes varying in size and number with the diameter of the roller.

The following formulæ shall be used in proportioning rollers:

For greatest total loads, including impact, with draw at rest,

$$p = 600d;$$

for loads with draw in motion,

$$p = 200d,$$

where p is the permissible pressure in pounds per lineal inch of roller, and d is its mean diameter in inches.

In no case shall the roller be less than twelve (12) inches in diameter and seven (7) inches on face.

All rollers, and the faces of the upper and lower tracks which are in contact with the rollers, are to be turned smooth to the forms of right frustums of cones, the vertices of which intersect at the centre of the drum, so that the rollers will have perfect contact with the tracks throughout their travel around the entire circumference.

A bearing is to be turned in the centre of each roller for the radial rod, and oil-holes are to be provided on both the interior and the exterior ends of the rollers, so that these bearings can be kept well lubricated.

The outer ends of the radial rods are to pass through the rollers, and the inner ends are to attach to a circular plate fitting closely around the centre casting. These radial rods are to be provided with either turnbuckles or nuts for adjusting the position of the rollers. Only square sections are to be used for the rods, and each must contain at least one square inch of section. The end of the rod passing through the roller must be upset so as to provide a turned shaft for the latter at least one and one-half ($1\frac{1}{2}$) inches in diameter. The outer ends of these rods are to pass through a stiff steel ring of rolled or built channel section, which is to serve as a spacer for the rollers. These channels must be made wide, but not deep, and their section is to be commensurate with the size of the turntable. They are to be held away from the rollers by friction-washers on the rods.

On the inside of the rollers collars are to be forged and turned on the radial rods to hold the said rollers in exact position on same. Turned bosses must be provided on both the inner and the outer ends of the rollers, to bear against the collars and the friction-washers.

An inner spacing-ring, of size commensurate with the magnitude of the drum, is to be attached to the radial rods. For large drums this should be in the form of a small curved plate girder lying in a horizontal plane and rigidly braced to the centre casting by radial struts that are riveted at the outer ends to the curved girder and at the inner ends to a large circular plate which fits snugly around a turned bearing on the centre casting. With this detail the radial rods are to be dispensed with, and in their stead are to be substituted heavy square bars, having their outer ends detailed as described for the radial rods, and their inner ends attached to the circular girder so as to hold the bars in a position exactly radial to the drum. These bars should not be less than two and a half (2½) inches square, and the journals should not be less than three (3) inches in diameter. There must be nuts at both ends of the bars so as to move the rollers in a radial direction, and the inner ends of the bars are to be so attached to the circular plate as to permit of the correction of any slight variation of their axes from a truly radial direction.

The centre casting must be made strong and heavy, and must be effectively anchored to the top of pier by eight (8) or more anchor-bolts not less than one and one-fourth (1¼) inches in diameter and not less than three (3) feet long. These bolts are to be made of soft steel, with cold-pressed threads and hexagonal nuts at top, and with split ends and wedges at bottom. The least allowable thickness of metal for this casting shall be one and one-half (1½) inches. The base shall be true and level; and an even bearing shall be secured by bedding in neat, Portland-cement mortar. For heavy draws this centre casting is to be set well into the masonry, then grouted in place.

All bearings for plates which rotate on this casting are to be turned smooth, and are to be provided with suitable oil-grooves, so they can be easily oiled.

Spans resting on drums of small diameter in proportion to the span length are to be anchored to the pivot-pier by means of a large anchor-rod in centre of pier, extending down ten (10) or fifteen (15) feet into same. This rod shall pass through the centre casting and through a box girder over the centre of the drum, which girder shall rivet into either the transverse or the longitudinal girders. The lower end of the rod shall pass through a heavy cast-iron anchor-piece embedded in the concrete of the pier. Both ends of the rod shall be provided with nuts for adjustment, and all details shall be made strong enough to develop the full strength of the anchor-rod. The upper nut shall be almost, but not quite, in contact with a large washer-plate that rests on the box girder. The size of the anchor-rod is to be determined by assuming an unbalanced upward wind load of five (5) pounds per square foot on the total area of the horizontal projection of one arm of the span.

The cap-plate for holding down the top connection-plate for the radial struts is to be attached to the top of the centre casting by means of a bolt tapped into same. This bolt is to be at least one and one-quarter ($1\frac{1}{4}$) inches in diameter.

The rack for turning the span is to be made in short sections, not over four feet long, so that in case of breakage only a small portion of the rack need be replaced. These rack segments are to be bolted to the lower track with tap-bolts not less than fifteen sixteenths ($\frac{15}{16}$) of an inch in diameter, and spaced not to exceed fifteen (15) inches between centres. There must be enough of them in any case in any one segment of the track to resist, with a good margin for contingencies, the entire shear (including that due to the rotating moment) caused by the effort of the pinion or pinions that engage with said segment. The least allowable thickness of metal in the rack shall be one and one-eighth ($1\frac{1}{8}$) inches. The ends of the rack segments are to be planed so as to secure close contact, and the abutting ends are to be bolted together with turned bolts at least seven eighths ($\frac{7}{8}$) of an inch in diameter.

The bottom of the rack and that portion of lower track upon which the rack bears are to be planed smooth. The width of the base of the rack shall be at least two thirds (⅔) of its height; and ribs bracing the vertical portion to the base shall be provided at distances not exceeding eighteen (18) inches.

Drainage-holes not less than three fourths (¾) of an inch in diameter, spaced not more than two (2) feet between centres, shall be bored in the lower-track segments, starting just back of the rack and leading to the outside of the track.

The girders over the drum shall be so arranged as to distribute the load over it properly. The number of bearing points required will depend upon the length of span, the distance from centre to centre of trusses, the total load to be carried, and the economical size of pivot-pier. The arrangement of the supporting girders in turn depends upon the number of bearing points to be used. For ordinary single-track bridges up to three hundred (300) feet in length a very good arrangement of girders over drum is secured by making the diameter of the drum and the length of centre panel equal to the distance from centre to centre of trusses; then the middle points of both the longitudinal and the transverse girders will be directly over the web of the drum, thus furnishing four points of bearing. Four more points of bearing are secured by putting in short diagonal girders, which connect to both transverse and longitudinal girders and bear on the drum at their centres. This arrangement gives in all eight (8) points of support.

The longitudinal, transverse, and diagonal girders over the drum shall be so designed that their rigidities will be such that when deflected under the load the extreme fibre-stress will be about the same in all the said girders.

The bottom-chord stresses in the centre panel can either be carried by the longitudinal girders, or the bottom-chord sections can be continued through the centre panel, the longitudinal girders being placed above them, and steel chairs being inserted beneath their centres to furnish bearings on the drum. In case that the bottom-chord stresses are carried by the longitudinal girders, ample provision must be made for them, as well as for the bending stresses, in designing the sections for these girders. Where the clearance over the waterway will permit, metal can be saved by letting the top flange of the longitudinal girder form the bottom chord of the truss.

In any arrangement of girders over the drum, bearing-plates at least one (1) inch thick must be used between the top flange of the drum and the bottom flanges of the girders, in order to make the points of concentration well defined, and so as to transmit the load properly from girders to drum.

All girders bearing on the drum are to have stiffeners on both sides of their webs at all points of concentration; and in no case are the stiffeners to be crimped, but are to have fillers beneath. They must have close bearings at top and bottom flanges, and are to be proportioned in the same manner as previously specified for those on the drum.

The rollers, tracks, drum, and girders over drum shall be completely assembled in the shop before shipment, all holes being reamed to fit and the sections being match-marked. Every roller must have a true bearing on both the upper and the lower tracks during a complete revolution of the draw.

Before the assembling of the rollers is done there must be marked on both the upper and the lower track segments a circle of the same diameter, which circles will come a trifle inside of the exterior ends of all rollers; then, after the turntable is perfectly adjusted, each roller is to be marked where these circles touch it. After the turntable is disconnected each roller is to be set up properly in a lathe, and the exterior periphery is to be chamfered off exactly to the points marked, so that when the turntable is set up in the field, if the exterior of each roller is brought exactly to the circles on the two tracks, the rollers will all be in their proper positions. These lines on the tracks will serve also afterwards to line up the rollers whenever the turntable is to be adjusted.

MACHINERY FOR TURNING THE SPAN AND LIFTING THE ENDS OF SAME

POWER.

When a draw-span is to be opened frequently, some kind of mechanical power must be used. The kind of power best adapted to any particular span depends upon a number of conditions, more especially the location of the bridge.

A gasoline-engine is an economic and convenient form of power for small spans which do not require more than twelve (12) or fifteen (15) horse-power to operate.

Duplicate electric motors, where direct connections can be made with electric-light or street-railway power-plants, are very efficient, convenient, and reliable; but in no case is it safe to depend upon storage-batteries for power. The use of electric motive power is therefore confined to bridges located in or near towns or cities.

Where over twelve (12) or fifteen (15) horse-power is required for operating the spans, and where electrical connections cannot be made, the steam-engine is the best form of power to use, except possibly in some special cases where water-power can be had conveniently.

Except in the case of short, light drawbridges, whenever mechanical power is employed it is necessary to apply the same to the rack by two pinions located diametrically opposite each other. If with this arrangement the tooth-pressure be still too high, it will be necessary to replace each pinion by a pair of pinions located as clos together as practicable. With pinions located far apart ome kind of an equalizer must be employed to divide the work equally between them, on account of the unavoidable, slight irregularities in the tooth-spacing of the entire rack. When electrical power is adopted, the equalizing may be done by means of electrical connections between the duplicate motors; but with any other power a mechanical equalizer between the two radial shafts must be employed. There will be no equalizing needed between the two pinions of each pair, on account of their being placed so close together.

With the equalizing arrangement just specified, it is legitimate to assume an equal division of work among all the pinions that engage the rack.

No matter what mechanical power be used, all spans must be provided also with hand-operating machinery.

METHOD OF DETERMINING POWER REQUIRED FOR OPERATING THE SPAN AND LIFTING THE ENDS.

The power required for turning any span is to be determined by the following formula:

$$\text{(1)} \qquad \text{H.P.} = \frac{.0125\,Wv}{550}.$$

where W = total load on rollers in pounds, and v = velocity on pitch-circle of rack in feet per second. The value of v is to be determined by the formula

$$v = \frac{\pi D}{4t},$$

where D = diameter of pitch-circle of rack, and t = assumed time in seconds for turning the draw through one fourth ($\frac{1}{4}$) of a revolution. This method gives the power required under ordinary conditions; but it is always necessary to figure also the power required to open the span against an assumed unbalanced wind-pressure. This is to be determined as follows:

The unbalanced wind-pressure on one arm is to be taken at five (5) pounds per square foot of the exposed surface of the floor and both trusses.

Let P = total unbalanced wind load on one arm in pounds, and v = velocity of travel of its centre of pressure in feet per second; then

$$\text{(2)} \qquad \text{H.P.} = \frac{Pv}{550}.$$

The value of v is to be determined by assuming a certain time t, in seconds, for turning the draw one fourth ($\frac{1}{4}$) of a revolution. Let l = distance in feet of the centre of pressure on one arm from the centre of the drum; then

$$(3) \qquad v = \frac{\pi l}{2t}.$$

For *Mechanical-power Turning-machinery* the greatest H.P. required is to be determined as follows:

Case I.—(*a*) By Formula (1) determine the H.P. required for turning the span in the least time in which it is probable that the said span will ever need to be opened.

Case II.—(*a*) By Formula (1) determine the H.P. required for turning the span in twice the time assumed in Case I. (*b*) By Formula (2) determine the H.P. required for operating the draw against the unbalanced wind load in twice the time assumed in Case I, and add together the two amounts of H.P. determined by (*a*) and (*b*). The sum will be the greatest H.P. required for Case II.

The greatest pressure on the teeth and torsion on shafts found for these two cases ~~is~~ are to be used, the metal being strained on the extreme fibre as hereinafter specified; but the said teeth and shafting must also be figured on the assumptions that the entire available capacity of the machinery is required merely to hold the draw from turning under an excessive unbalanced wind-pressure, and that under these conditions the metal is strained twice as high as hereinafter specified.

For *Hand Turning-machinery* the H.P. required to turn the span in the *least time* in which it is probable that it will ever need to be opened by man-power is to be found by the formula previously specified; then the number of men required to perform this work is to be determined by assuming that six (6) men are equivalent to one H.P. In proportioning all parts of the hand-operating machinery there shall be assumed on the levers as many men as are required by the above method, each man exerting a horizontal thrust of one hundred and twenty (120) pounds. Under such conditions the metal is to be strained the same as hereinafter specified for machinery operated by mechanical power under ordinary conditions.

DETAILS OF MACHINERY.

OPERATING MACHINERY.

All gear-wheels are to be of cast steel with cut gears. To determine the size of any gear-wheel, the tooth-pressure on the pitch-circle is first to be found as follows:

For gears moved by mechanical power only,

$$P = \frac{550\,H.P.}{v},$$

where $H.P.$ = horse-power to be transmitted by gear, v = velocity in feet per second at its pitch-circle, and P = tooth-pressure.

For gears moved by hand-power,

$$P = 120NM,$$

where N = number of men, M = multiple of lever over gear under consideration, and P = tooth-pressure.

Having thus determined the tooth-pressure, the pitch can be found by the following formula:

$$p = .025\,\sqrt{\tfrac{2}{3}P},$$

for gears in which the face is equal to $2\frac{1}{2}$ times the pitch, where p = pitch, and P = total tooth-pressure.

This allows an extreme fibre-stress on the teeth of eight thousand (8,000) pounds per square inch, which is to be the standard intensity for all teeth under ordinary conditions of operation. Bevel-gears are to be considered as only three fourths ($\frac{3}{4}$) as strong as spur-gears of the same pitch and face. The use of bevel-gears with very thin teeth will not be allowed, even though they be of standard pattern; but special bevel-gears with thicker teeth than usual will have to be manufactured.

All gears are to be key-seated and finished in accordance with the practice of the best machine-shops. All pinions gearing into the rack and into the large spur-wheels are to be shrouded on top, and the extra strength obtained by this shrouding is not to be counted upon in proportioning the size of the teeth of the pinion.

All shafting is to be of cold-rolled steel, and is to be provided with couplings, collars, and keys for gears.

All couplings must be strong enough to develop the full strength of the shafting, and must be keyed to the same, flange-couplings being preferred. All couplings are to be placed as near the bearings as practicable.

Suitable collars are to be used wherever they are necessary to hold the shafting from moving longitudinally.

The greatest allowable length of any shaft between centres of bearings is to be determined by the formula

$$L = 75 \sqrt[3]{d^2},$$

where $L =$ the unsupported length in inches, and $d =$ diameter of shaft in inches.

The diameter required for any shaft is to be determined by the following formula:

$$d = 4\sqrt[3]{\frac{H.P.}{N}},$$

where $d =$ diameter required, $H.P =$ the horse-power to be transmitted, and $N =$ the number of revolutions per minute. This will allow for all bending that will come on any well-designed and properly supported shaft under ordinary conditions, and provides for an extreme fibre-stress of about twelve thousand (12,000) pounds per square inch, under the assumption that the twisting moment and the bending moment are about equal.

Every shaft, however, after being designed by the preceding formula must be checked as follows, and if found weak must be properly strengthened either by increasing the diameter or by reducing the lever arm or arms of the bending moment.

First, find the twisting moment and the bending moment (including that caused by the weight of the shaft itself) by computing the tooth-pressure, which is the force producing directly these moments, calling the twisting moment T and the bending moment M. The equivalent twisting moment for a combination of these two moments is given by the equation

$$T' = M + \sqrt{M^2 + T^2},$$

where T' is the equivalent twisting moment.

The corresponding extreme fibre-stress is to be found by the equation

$$f = 5.1\frac{T'}{d^3},$$

where d is the diameter of the shaft, and f is the extreme fibre-stress. This should never exceed twelve thousand (12,000) pounds per square inch for all ordinary conditions of operation, or twenty-four thousand (24,000) pounds per square inch for the unusual conditions of the machinery stalled by the unbalanced wind-pressure when working at its utmost capacity.

In no case is any shaft of less than two and one-quarter ($2\frac{1}{4}$) inches in diameter to be used for any part of the machinery of draw-spans.

Suitable cast-iron boxes are to be provided for all bearings. All boxes, bearings, couplings, collars, etc., are to be made in accordance with the best machine-shop practice. The boxes for the line of shafting running to ends of span are to have wooden shims beneath them so that the shaft can be aligned perfectly after the span is swung.

The hand-power turning-machinery is to be so arranged that the levers can be conveniently applied to shafts near the centre of span for both the turning and the end-lifting machinery. Shafts must also be provided for applying the hand-power levers to the end-lifting machinery at each end of the span. Suitable hand-levers are to be provided for as many men as are required for operating the draw. These levers are to be constructed entirely of steel, excepting only the small wooden quarter-rounds at the ends by which the men take hold.

All machinery shall be so arranged that the span can be turned completely around in either direction, and so that it is reversible in every particular.

END-LIFTING APPARATUS.

The ends are to be lifted and locked by means of a toggle mechanism to be operated by screws at each end of the span. The entire machinery is to be made strong enough, with the previously specified intensities of working-stresses, to exert an upward force on each end of each truss equal to the assumed uplift in case of mechanical power; or to transmit to the end rollers the greatest force that the men can exert on the hand levers, assuming that as many men will be applied thereto as are required for the turning-machinery, and that each man exerts a horizontal thrust of one hundred and twenty (120) pounds, straining the metal the same as in the case where the power is mechanical.

In case of mechanical power, all the teeth and shafting must also be figured on the assumption that the entire available capacity of the machinery is required merely to start motion, and that under this condition the metal is strained twice as high as herein specified.

The size of screw required is to be determined by the following formula:

$$d = .02\sqrt{P},$$

where d = diameter of screw at base of threads, and P = axial pressure on screw.

The axial pressure is to be determined for the two following cases, the greater pressure thus found being adopted:

Case I.—

$$P = \frac{2Rh'}{h},$$

where R = total assumed upward reaction at one end of span, h' = greatest rise of ends when end lifts are applied, and h = travel of nut on screw necessary to produce the rise h'.

The factor two (2) is used to allow one hundred (100) per cent for friction.

Case II.—

$$P = 80MN,$$

where M = the number of pounds pressure the screw will exert for one pound applied on the lever, and N = number of men on said lever. By using eighty (80) instead of one hundred and twenty (120) in the above formula, there is made an allowance of thirty-three and a third ($33\frac{1}{3}$) per cent for friction, which is certainly lower than it will ever be under ordinary working conditions.

Assuming the coefficient of friction low in this case makes an error on the side of safety.

For all ordinary conditions, Case II will give the greater value for P. The threads are to be standard square threads, and the nuts which work on them are to be made long enough to keep the greatest working unit pressure on said threads down to five hundred (500) pounds per square inch.

All links used in the toggle mechanism are to be proportioned by the formula

$$p = 10,000 - \frac{300l}{t},$$

where l = greatest unsupported distance between fillers, except in links in which only one filler is used between two flats, when it is to be taken as the entire distance from centre to centre of end-pins, t = thickness of each link, and p = the intensity of working compressive stress.

In no case is the diameter of any pin used in a toggle to be less than two and a half ($2\frac{1}{2}$) inches.

Rail lifts are to be provided in connection with the end-lifting toggle, and the mechanism therefor is to be so designed that the rails will not begin to rise until the end rollers have been drawn from their bearings on the end shoes. The rails shall be lifted so as to clear by one (1) inch all parts over which they must pass in turning, under the assumption that the temperature of the top chords is higher by thirty (30) degrees Fahrenheit than that of the bottom chords.

Suitable guide-chairs for the rails near the ends of the span are to be provided beneath the same on at least fifteen (15) ties from each end of the span. These chairs must be either spiked or bolted to the ties, and must hold the rails firmly in place. Guide-rods such as are employed in ordinary switch-work are to be used every six feet between the portions of the rails resting in the guide chairs.

The strength of all parts of the rail-lifting machinery is to be determined by computing the force necessary to deflect the two rails the required amount in a distance of twenty (20) feet, and adding fifty (50) per cent thereto for friction.

If considered necessary for any particular span, latches are to be provided for holding the ends in place; but under ordinary conditions the track-rails and the end rollers are all that will be required.

In double-track drawbridges special attention must be paid to the designing of not only the lifting-gear, but also the trusses themselves, in order to ensure that, under the most unfavorable circumstances possible, there shall be no lifting of the ends of trusses off their supports. If such a lifting were possible, the result would certainly be the derailment of an entering train, and consequently disaster to the span. To prevent such uplifting the trusses must be deep and very rigid, and the lift of the ends must be from one (1) to two (2) inches, according to the length of the span.

SHOES AND END-BEARING ROLLERS.

Rollers are to be provided beneath the end-pins of trusses and attached to the span by means of links which form a part of the toggle. The rollers must be bored so as to fit over the pins at the bottom of the links. Both the pins and the inside of the rollers must be finished very smooth; and provision must be made for oiling the bearings between them. The allowable intensity for bearing between rollers and pins shall be ten thousand (10,000) pounds per square inch of horizontal projection of pin inside of the roller.

No roller shall be less than six (6) inches in diameter, and the pins inside of same shall not be less than three and seven-sixteenths ($3\frac{7}{16}$) inches net in diameter. The play between rollers and their pins shall not be over one thirty-second ($\frac{1}{32}$) of an inch. The links forming the support for the ends of trusses are to be proportioned by the formula

$$p = 10{,}000 - 300\frac{l}{t},$$

where p = intensity of working compressive stress, l = greatest unsupported length of one link, and t = thickness of same.

In all drawbridges where, on account of infrequent operation combined with great changes in temperature and great length of arms, there is a tendency to drag the rollers longitudinally on their bearings, the detailing of the link supports must be such as to provide sufficient rigidity to overcome the friction of the rollers on their bearings, and thus permit the lifting apparatus to accommodate itself to extreme changes of temperature without overstraining any of its parts.

The bearings for rollers on the shoes shall be cupped one-eighth ($\frac{1}{8}$) inch or more in depth so as to provide ample bearing area, using an intensity of ten thousand (10,000) pounds, impact being included in the calculated load. The shoes to receive the end rollers may be made of either cast or structural steel, and are to be anchored firmly to the masonry. The two shoes at one end of span are to be connected to each other by means of adjustable rods not less than one and one-half ($1\frac{1}{2}$) inches in diameter, and strong enough to take up the entire thrust from the toggle.

Shimming-plates varying in thickness from one fourth ($\frac{1}{4}$) to one half ($\frac{1}{2}$) of an inch and of a total depth of not less than three (3) inches are to be used beneath the shoes so as to provide adjustment for the ends of the span.

Shoulders must be provided on the shoes to furnish a bearing for the rollers when they are lowered by the toggle. Each shoulder must be turned so as to fit the roller exactly, when the axis of the pin through the said roller is in the vertical plane of the truss. The height of these shoulders above the bottom of the rollers shall be about one third of the diameter of said rollers, but never enough to involve the possibility of collision with the draw-span during its revolution and when the top chords thereof are thirty (30) degrees Fahrenheit warmer than the bottom chords.

All parts of the end-lifting machinery must be finished in accordance with the best machine-shop practice, and all sliding surfaces shall be provided with oil-holes that are easily accessible.

In all cases end floor-beams with double webs shall be used, in order to provide proper support for the end-lifting machinery.

Whenever spans are to be floored for highway traffic, all keyholes for applying hand-levers are to be provided with suitable cast-iron caps.

Whenever practicable, the end-lifting toggle machinery is to be assembled in the shops to make sure that it will work satisfactorily.

HOUSES AND SUPPORTS.

Wherever mechanical power of any kind is to be used for operating any draw-span, a suitable house is to be provided for same. The size of the house required will depend upon the kind of power to be used, and the amount thereof. All parts of the house shall be durable and strong, and shall be finished in a first-class and workmanlike manner. A sufficient number of windows is to be put in to light properly all parts of the building. The house shall be placed high enough in the tower to give the required clearance beneath its supports, and, where shallow trusses are used, it shall be placed entirely above the span. The supports for the house shall be designed to carry the weight of the latter and that of all machinery to be placed therein, together with a proper allowance for live load. In general, steel beams shall be used for the joists supporting the floor, and all parts of the latter shall be made strong enough to carry three hundred and fifty (350) pounds per square foot.

The weight of the house and its machinery must always be considered in proportioning all parts of the structure which will be affected by these loads, whether the span is to be provided with mechanical power at first or not, as it may become necessary later on to put it in. The wind load on the house must also be considered in proportioning the tower posts and all bracing between them.

CAMBER AND DEFLECTION.

The lengths of all truss members shall be such that when the assumed uplift is applied at the ends of the span, and when the greatest live load is on the structure, the centre lines of the bottom chords from end to end of span will lie in a horizontal plane. The vertical movement of the ends, from the condition of no stress in the chords, when the weight of the finished span is supported on the falsework, to the condition of the span swung, must be very carefully figured, as upon this will depend the camber increments or decrements in lengths of members, the clearances, adjustments, etc.

CHAPTER XVI.

GENERAL SPECIFICATIONS GOVERNING THE DESIGNING OF STEEL HIGHWAY BRIDGES AND VIADUCTS.

GENERAL DESCRIPTION.

CLASSIFICATION.

HIGHWAY bridges shall be divided into three classes, viz., Class A, which includes those that are subject to the *continued* application of heavy loads; Class B, which includes those that are subject to the *occasional* application of heavy loads; and Class C, which includes those for ordinary, light traffic.

In general it may be stated that bridges of Class A are for densely populated cities; those of Class B for smaller cities and manufacturing districts; and those of Class C for country roads.

MATERIALS.

All parts of the structure, excepting the flooring or paving shall, for all spans of ordinary lengths, be of medium steel excepting only that rivets and bolts are to be of soft steel, and adjustable members of either soft steel or wrought iron. For very long spans high steel may be used for top chords, inclined end posts, pins, and eye-bars in bottom chords and in main diagonals of panels where there is no reversion of stress when impact is included. Cast iron will not be allowed to be used in the superstructure of any highway bridge or trestle, except for purely ornamental work, cast steel being employed wherever important castings are necessary.

JOISTS, PLANKS, GUARD-TIMBERS, AND WOODEN HAND-RAILS.

Joists, planks, guard-rails, hand-rails, and all other timber portions of the structure shall be of long-leaf, Southern, yellow pine, or other timber which, in the opinion of the Engineer, is equally good and serviceable.

The sizes of the timber joists shall be such as to give the requisite resistance to bending, the effect of impact being considered; but no joist shall be less than three (3) inches wide or twelve (12) inches deep.

As a rule the depth of a joist shall not exceed four (4) times its width. Otherwise, the joists shall be properly bridged at distances not exceeding eight (8) feet.

They shall be proportioned by the formula

$$M = \frac{1}{6} Rbd^2,$$

where M is the greatest bending moment in inch-pounds upon a joist, R is the intensity of working-stress in pounds, b the width of the joist in inches, and d the depth of same in inches.

Joists shall be dapped at least one-half ($\frac{1}{2}$) inch upon their bearings, and shall have their tops brought to exact level before the planks are laid thereon.

They shall be spaced not to exceed two (2) feet between centres; shall, preferably, lap by each other so as to extend over the full width of the floor-beam; and shall be separated half an inch, so as to permit the circulation of air. The outside joists, however, shall abut so as to provide flush surfaces from end to end of span.

Floor-planks for the main roadway shall be at least three (3) inches thick and from eight (8) to ten (10) inches wide, and shall be laid with one-quarter ($\frac{1}{4}$) inch openings. Each plank shall be spiked to each joist on which it rests by two (2) seven (7) inch cut spikes, the holes for which shall be bored in order to avoid splitting the timber, or else by two (2) seven (7) inch wire nails.

Whenever a wearing-floor is used, the lower planks must be planed on the upper side and sized to a uniform thickness, and the wearing-floor must be planed on the lower side so as to ensure a perfect bearing between upper and lower floors.

Floor-planks for footwalks shall be at least two (2) inches thick and not much more or less than six (6) inches wide, and shall be laid with one-half ($\frac{1}{2}$) inch openings. Each of said planks shall be spiked to each joist upon which it rests by two (2) six (6) inch cut spikes, the holes for same being bored.

All planks shall be laid with the heart side down.

There shall be a wheel-guard of a scantling not less than four (4) inches by six (6) inches on each side of the roadway to prevent wheel hubs from striking the trusses. It is to be laid on its flat, and blocked up from the floor by shims at least one (1) foot long, six (6) inches wide, and two (2) inches thick, spaced not more than seven (7) feet between centres, each shim being spiked to the floor by four (4) four-and-a-half ($4\frac{1}{2}$) inch cut spikes. The guard-rails are to be bolted to the floor through the centre of each shim by a three-quarter ($\frac{3}{4}$) inch bolt, which must also pass through the joist beneath. When the guard-rails are bolted to the wooden hand-rail posts, the bolt-heads are to be countersunk into the guard-rail, so as to make a flush surface on the inner face of same. The joints in the guard-rail are to be lap-joints, at least six (6) inches long, each located symmetrically over the middle of a shim. When a bridge is on a heavy grade, the inner, upper corners of the guard-rails are to be covered with steel angles fastened to the timber by countersunk screws, spaced about eighteen (18) inches apart, so as to protect the guard-rails from the injurious effects of using them instead of wheel-brakes for heavily-loaded wagons.

When wooden hand-rails are employed, they are to be made of pine, the posts being $4'' \times 6'' \times 4' 6''$ to $5'$, with two (2) runs of $2'' \times 6''$ timbers—one on its flat and the other below on edge to support the first for a hand-rail—and one (1) run of $2'' \times 12''$ hub-plank.

The posts are to be spaced not to exceed ten (10) or, preferably, eight (8) feet apart. The hand-railing is to be firmly attached to the bridge, and rigidly braced. When the rigidity of a hand-railing is dependent upon that of the outer joists, the latter must be properly bridged and stiffened. Any other wooden hand-railing of equal strength and rigidity, and which is satisfactory to the Engineer, will, however, be accepted.

When iron hand-railing is employed, it is to be of a firm, substantial pattern, pleasing to the eye, and rigidly attached to the trusses or floor-beams. Both through and deck bridges are to be provided with a hand-rail on each side, not less than three and a half (3½) feet high above the floor. In case there be any liability of a horse jumping over this railing, its height must be increased to four and a half (4½) or five (5) feet. There must be a hand-rail on the outside of each sidewalk, not less than three and a half (3½) feet in height above the floor.

FLOORING ON APPROACHES.

All floor-timbers, guards, and railings shall extend over all piers and abutments, and make suitable connection with the embankments at the ends of the structure. Aprons or cover-joints of steel plate shall be provided at the ends of spans, if required. The floors of the sidewalks shall extend to and connect with the floor of the main roadway, so as to leave no open space between them.

STREET-RAILROAD TRACKS.

Should there be one or more street-railroad tracks crossing the bridge, there must be directly under each rail a joist or stringer, properly proportioned to resist the effect of the total maximum load on the rail; and the bending effect of the concentrated loads upon the floor-beams must be duly considered.

The rails shall be so laid as to offer as little obstruction as possible to the wheels of vehicles.

PAVED FLOORS.

Where paved floors are adopted, the pavement shall be of the best of its kind, and shall be built according to the latest and most approved specifications. Paved floors are always to be supported by steel stringers, preferably of rolled I beams, spaced generally not to exceed three (3) feet six (6) inches between centres. For asphalt or stone-block pavements, a buckled-plate floor, with concrete thereon, shall be used. The surface of the pavement must be thoroughly drained so as not to retain water, and the upper surface of the buckled plate, before it is covered with the concrete, must be protected from rusting by a liberal use of the best obtainable preservative coating.

When wooden-block paving is adopted, it may rest on a timber floor from four (4) to five (5) inches thick, which in turn rests on and is spiked to timber shims that are bolted effectively to the steel stringers.

All paved floors must be pitched so as to drain transversely to the structure; but plank floors need not be pitched, as the water will drain through the quarter-inch openings.

CLEARANCES.

The smallest allowable clear roadway shall be twenty (20) feet, measured between inclined end posts, excepting for cheap country bridges, where it may be reduced to eighteen (18) feet, or even to fourteen (14) feet, in case that the bridge be so short that no provision need be made for teams passing thereon.

The smallest allowable clear headway shall be fourteen (14) feet, except for bridges in cities where the ordinances require a greater height. The corner brackets may, however, encroach on the specified clear headway, provided they do not extend either laterally or downward more than five (5) feet.

EFFECTIVE LENGTHS AND DEPTHS.

See Specifications for Railroad Structures.

STYLES OF BRIDGES FOR VARIOUS SPAN LENGTHS.

In general, spans of and below twenty (20) feet are to consist of rolled beams or simply wooden joists; spans from twenty (20) to sixty (60) feet, of plate girders, spans from sixty (60) to ninety (90) feet, of open-webbed, riveted girders of single cancellation, or pin-connected "A" trusses; and spans exceeding ninety (90) feet, of pin-connected trusses.

The use of pony truss bridges of any kind is prohibited, excepting only half-through, plate-girder spans, in which the top flanges are held rigidly in place by brackets riveted to cross girders that are spaced generally not to exceed fifteen (15) feet apart.

FORMS OF TRUSSES.

The forms of trusses to be used are as follows:

For pin-connected spans up to ninety (90) feet, the "A" truss.

For open-webbed, riveted girders, the Warren or Triangular girder, with verticals dividing the panels; also the Pratt truss.

For deck-spans carrying joists on the top chords, the Warren or Triangular girder with verticals dividing the panels of the top chords.

For spans between ninety (90) feet and about two hundred and fifty (250) feet, Pratt trusses with top chords either straight or polygonal.

For spans exceeding two hundred and fifty (250) feet, Petit trusses.

It is understood that these limiting lengths are not fixed absolutely, as the best limits will vary somewhat with the width of bridge and the live load to be carried.

MAIN MEMBERS OF TRUSS-BRIDGES.

All spans of every kind shall have end floor-beams, riveted rigidly to the trusses or girders, for supporting the joists or stringers.

Steel stringers are, preferably, to be riveted to the webs of the cross-girders, but wooden joists are generally to rest on top of the latter.

In general, all trusses shall have main end posts inclined, All trusses shall be so designed as to admit of accurate calculations of all stresses, excepting only such unimportant cases of ambiguity as occur when two stiff diagonals are used in a middle panel.

In important bridges with steel stringers, all lateral bracing and other sway-bracing shall be rigid above and below : i.e. the sections must be capable of resisting compression, adjustable rods for such bracing being allowed only in towers of draw-spans and in the lower lateral systems of deck-bridges ; but, in cheap country bridges, the lateral and other sway diagonals may be adjustable rods.

The stiff diagonals of lower lateral systems, which shall be of double cancellation, shall be riveted rigidly to all the steel stringers where they cross them.

In the trusses of important bridges counterbracing the web shall be effected by using stiff diagonals, but in cheap bridges it may be done by using counters of adjustable rods.

All through-spans shall have portal bracing at each end, carried as low as the specified clear headroom will allow. The portal struts shall be riveted rigidly to the web or both flanges of the inclined end posts. Riveting portals to one flange only will not be allowed.

When the height of the trusses is great enough to permit, transverse, vertical sway-bracing shall be employed ; otherwise, corner brackets of proper size, strength, and rigidity are to be riveted between the posts and the upper lateral struts.

Deck-bridges shall, as a matter of precaution, have sway-diagonals between opposite vertical posts of sufficient strength to carry one half of a panel-truss live load with its impact allowance ; and the transverse bracing between the vertical or inclined posts at each end of span shall be sufficiently strong to transmit properly to the masonry one half of the total wind-pressure carried by the upper lateral system of the span.

The lower lateral systems of deck-bridges may be made of adjustable rods in alternate panels, thus leaving every other panel unbraced, and forcing the wind-pressure from below up the vertical bracing and to the ends of the span by the upper lateral system.

In important bridges, suspenders or hip verticals and two or more panel lengths of bottom chord at each end of span shall, preferably, be made rigid members, except that eye-bars are to be used for bottom chords of "A" truss bridges.

All floor-beams are to be riveted to the truss-posts in truss-spans, excepting in the case that eye-bars be used for suspenders or hip verticals. In such cases floor-beam hangers may be used, provided they be made of plates or shapes, and that they be stayed at their upper ends against all possibility of rotation.

CONTINUOUS SPANS.

See Specifications for Railroad Structures.

TRESTLE-TOWERS.

In general, the descriptive specifications for railroad trestles are to be followed in designing highway trestles or viaducts, except that in cheap structures all sway-diagonals of towers may be made of adjustable rods, with horizontal struts at the panel points, provided that the struts be rigidly riveted to the columns.

CAMBER.

All trusses must be provided with such a camber that, with the heaviest live load on the span, the total camber shall never be quite taken out by deflection. With parallel chords, sufficient camber will be obtained by making the top-chord sections longer than the corresponding bottom-chord sections by five thirty-seconds ($\frac{5}{32}$) of an inch for each ten (10) feet of length.

Plate girders and shallow, open-webbed, riveted girders should not be given any camber.

EXPANSION, ANCHORAGE, AND NAME PLATES.

See Specifications for Railroad Structures.

LOADS.

The loads to be considered in designing highway bridges and trestles are the following; and all parts of same are to be proportioned to sustain properly the greatest stresses produced thereby for all possible combinations of the various loads, excepting only that the live load and wind load cannot act together, unless the structure carry an electric railway; for the reason that no person would venture on the bridge when even one half of the assumed wind-pressure is acting.

A. Live Load.
B. Impact Allowance Load.
C. Dead Load.
D. Direct Wind Load.
E. Indirect Wind Load or Transferred Load.
F. Effects of Changes of Temperature.

When a highway bridge carries an electric railway, it shall be proportioned also for—

G. Traction Load, and,
H. Centrifugal Load.

In calculating the stresses caused by a uniform moving load, the load shall be assumed to cover the panel in advance of the panel point considered; but the half-panel load going to the forward panel point will be ignored; or, in other words, the uniform load will be treated as if concentrated at the various panel points.

LIVE LOADS.

The uniformly distributed live loads per square foot of floor, including the entire clear widths of both main roadway and footwalks, shall be taken from the curve diagram shown on Plate V.

In applying these curves the span lengths used shall be as follows:

For stringers and joists, a single panel length; for floor-beams and single panel suspenders with their corresponding primary-truss struts, two (2) panel lengths; for hip verticals of Petit trusses, four (4) panel lengths; and for main-truss members, the length of span loaded when the member under consideration receives its maximum stress.

In the case of bridges with exterior sidewalks, one sidewalk only and the roadway are to be considered loaded when proportioning the beam-hangers and *primary* truss members of all bridges, and when proportioning the main-truss members of all spans less than one hundred (100) feet for bridges of Class A, and of all spans less than eighty (80) feet for bridges of Classes B and C. In all other cases both of the sidewalks and the roadway are to be considered loaded. The eccentric loading increases the live load per truss. But, when a bridge has only one exterior sidewalk, the effect of the eccentric loading is to be considered to act upon the whole of the nearer truss, and the sidewalk is to be considered empty when calculating the stresses in the farther truss. Floor-beams of bridges with one or two exterior sidewalks are to be proportioned on the assumption that, first, the main roadway is loaded, and the sidewalk or sidewalks are empty; and, second, that the main roadway is empty, and the sidewalk or sidewalks are loaded, due account being taken of the effect of reversing stresses as hereafter specified.

In addition to the preceding loads, the floor, joists, floor-beams, beam-hangers, and primary-truss members are to be proportioned for the following concentrated loads, which are, however, supposed to occupy a whole panel length of the main roadway to the exclusion of the other live loads there (excepting only the electric-railway live load).

CLASS A.

A road-roller weighing thirty thousand (30,000) pounds, of which twelve thousand (12,000) pounds are concentrated upon the roller in front of the machine, and nine thousand (9000) pounds on each of the wheels at the rear, the distance between the central planes of these wheels being five (5) feet, and that between their axis and the axis of the front roller eleven (11) feet. The width of the front roller is to be four (4) feet, and that of each rear wheel one foot eight inches (1′ 8″).

CLASS B.

A concentrated load of sixteen thousand (16,000) pounds equally distributed upon two pairs of wheels, the axes of which are eight (8) feet apart, and the central planes of the wheels six (6) feet apart.

CLASS C.

A concentrated load of ten thousand (10,000) pounds distributed in the same manner as for Class B.

The road-roller load is assumed to be equally divided between all of the joists that it can cover, and the wheel loads for Classes B and C equally between two joists.

In case that the highway bridge or trestle carries a single-track line of electric road, that one of the four car or train loads, shown on Plate VI, which most closely approximates to the greatest railway load that will be carried by the structure is to be adopted, and is to be assumed to occupy ten (10) feet in width of the entire clear roadway of the span to the exclusion of all other live loads on said ten (10) feet, except as hereinafter specified for floor-beams and primary-truss members.

The equivalent uniformly distributed live loads, given by the curves on Plate VI, are to be used when making computations instead of the concentrations just specified.

The impact allowance for these electric railway loads is to be taken from the Specifications for Railroad Structures.

The floor system and primary-truss members are to be figured for these electric train loads when passing either the road-roller or the heavy wagon-load; and the trusses as a whole are to be figured for a uniform load found by combining the equivalent electric load, considering it to occupy ten (10) feet of roadway, together with its impact allowance, with the regular uniform live load per square foot of floor on the remaining width of clear roadway, together with its proper impact allowance, provided that the equivalent live load per lineal foot for the cars, plus the proper impact allowance, exceed the regular live load for a ten (10) foot width of roadway, plus its proper impact allowance. If it do not so exceed, the regular uniform live load is to be employed.

IMPACT-ALLOWANCE LOAD.

The impact-allowance load is to be a percentage of the uniform live load, found by the formula

$$P = \frac{10000}{L + 150},$$

where P is the percentage and L the length in feet of span or portion of span that is covered by the live load, when the member considered is subjected to its maximum stress.

DEAD LOAD.

See Specifications for Railroad Structures.

WIND LOADS.

For highway structures the wind loads per lineal foot of span for both the loaded and the unloaded chords are to be taken from the curves shown on Plate VIII.

This diagram was figured for a clear roadway of twenty (20) feet. For wider structures, the wind loads are to be increased two (2) per cent for each foot of width in excess of twenty (20).

The wind loads given on the diagram have been computed from detailed designs for simple spans up to seven hundred and fifty (750) feet in length, but beyond this limit they have been assumed; consequently, in designing spans of greater length than this, it will be necessary to check the assumed wind-pressure after the sections are proportioned, using an intensity of twenty-five (25) pounds per square foot.

The intensities employed in preparing the curves varied from forty (40) pounds for very short spans to twenty-five (25) pounds for very long ones.

For viaducts, the wind-pressure on the empty structure is to be assumed as three hundred (300) pounds per lineal foot on the spans at the level of the floor, and two hundred and fifty (250) pounds for each vertical foot of each entire tower.

The wind loads for longitudinal bracing are to be taken as seven tenths (0.7) of those for the transverse bracing.

For viaducts carrying electric trains, the wind loads are to be taken from the Specifications for Railroad Structures.

All wind loads are to be treated as *moving loads*.

INDIRECT WIND LOAD OR TRANSFERRED LOAD.

See Specifications for Railroad Structures.

TRACTION LOAD.

See Specifications for Railroad Structures.

CENTRIFUGAL LOAD.

See Specifications for Railroad Structures.

EFFECTS OF CHANGES OF TEMPERATURE.

See Specifications for Railroad Structures.

INTENSITIES OF WORKING-STRESSES.

See Specifications for Railroad Structures.

BEARINGS UPON MASONRY.

See Specifications for Railroad Structures.

REVERSING-STRESSES.

See Specifications for Railroad Structures.

NET SECTION.

See Specifications for Railroad Structures.

BENDING MOMENTS ON PINS.

See Specifications for Railroad Structures.

COMBINATIONS OF STRESSES.

The Specifications for Railroad Structures under this heading are to be followed, with this exception: in bridges and viaducts that do not carry trains, the live load and the wind load are assumed not to act simultaneously.

BENDING ON TOP CHORDS.

See Specifications for Railroad Structures.

BENDING ON INCLINED END POSTS.

The Specifications for Railroad Structures under this heading are to be followed, with this exception: in bridges that do not carry trains, the live load and the wind load are assumed not to act simultaneously.

BENDING DUE TO WEIGHT OF MEMBER.

See Specifications for Railroad Structures.

GENERAL LIMITS IN DESIGNING.

The following general limits shall be adhered to in designing highway bridges and viaducts:

The perpendicular distance between central planes of trusses shall never be less than one twentieth ($\frac{1}{20}$) of the span.

The length of any bracket cantilevered beyond a truss or girder shall never exceed one half of the perpendicular distance between the central planes of adjacent trusses or girders, unless there be more than two trusses to the span.

No metal less than five sixteenths ($\frac{5}{16}$) of an inch in thickness shall be used, except for filling-plates; and in important bridges this limit shall be increased to three eighths ($\frac{3}{8}$) of an inch.

The least allowable thicknesses of webs of rolled I beams shall be as follows:

24″ I beams	$\frac{9}{16}$″	webs.
20 " "	$\frac{1}{2}$	"
18 " "	$\frac{7}{16}$	"
15 " "	$\frac{3}{8}$	"
12 " "	$\frac{5}{16}$	"

No channel less than six (6) inches in depth shall be used except for lateral struts, in which five (5) inch channels may be employed.

No angles less than $2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{5}{16}''$ shall be used except for lacing.

No eye-bars less than three (3) inches deep or five eighths ($\frac{5}{8}$) of an inch thick shall be employed; and the depths of eye-bars for chords and main diagonals shall not be less than one sixtieth ($\frac{1}{60}$) of the horizontal length of same.

No adjustable rod shall have less than three quarters ($\frac{3}{4}$) of a square inch of cross-section.

The shortest span length for trusses with polygonal top chords shall be one hundred and sixty (160) feet.

The limit of span length in which steel stringers can be riveted continuously from end to end of span shall be two hundred (200) feet. Beyond this limit sliding-bearings must be used at one or more intermediate panel points; and in no span shall there be a length of continuously riveted stringers exceeding two hundred (200) feet.

For all compression-members of trusses and for columns of viaducts the greatest ratio of unsupported length to least radius of gyration shall be one hundred and twenty (120), excepting those members whose main function is to resist tension. In these the limit may be raised to one hundred and fifty (150).

The corresponding limit for all struts belonging to sway-bracing shall also be one hundred and fifty (150).

GENERAL PRINCIPLES IN DESIGNING ALL HIGHWAY STRUCTURES.

See Specification for Railroad Structures.

RIVETING.

In general, the specifications for riveting given for railroad structures shall apply also to highway structures, except that in the latter the diameters for rivets may be reduced to three quarters ($\frac{3}{4}$) of an inch for ordinary work.

DETAILS OF DESIGN FOR ROLLED I-BEAM SPANS.

Rolled I beams used as longitudinal girders shall have, preferably, a depth not less than one fifteenth ($\frac{1}{15}$) of the span. They shall be proportioned by their moments of inertia. The spacing shall generally not exceed three (3) feet six (6) inches.

Provided that wooden shims be bolted to the top flanges for spiking the planks thereto, no sway-bracing will be required; but otherwise it must be used. Each I beam is to have at each end a pair of stiffening angles, fitting tightly at both top and bottom to the flanges, to carry the load to the masonry and to form part of the end bracing-frames. Each pair of girders is to have a bracing-frame at each end; and under each end of each I beam there is to be riveted a bearing-plate of proper area and thickness (never less than five eighths [$\frac{5}{8}$] inch) to distribute the load uniformly over the masonry, said plate being bolted effectively to the latter, with due provision for expansion and contraction.

DETAILS OF DESIGN FOR PLATE-GIRDER SPANS.

In designing plate-girder spans for highway structures, the corresponding specifications for railroad structures are to be followed, except that the depths of girders shall preferably be not less than one twelfth ($\frac{1}{12}$) of their span, that metal five-sixteenths ($\frac{5}{16}$) inch thick may be used, and that the stiffening angles may be made as small as two and a half ($2\frac{1}{2}$) by two and a half ($2\frac{1}{2}$) inches.

DETAILS OF DESIGN FOR OPEN-WEBBED, RIVETED GIRDER SPANS.

See Specifications for Railroad Structures.

DETAILS OF DESIGN FOR PIN-CONNECTED SPANS.

The sections of top chords and inclined end posts of through-spans shall consist, generally, of two rolled or built channels and a single cover-plate. In the case of built channels, the section of the member must be so proportioned as to bring its centre of gravity as near as possible to the middle of the webs.

Main vertical posts shall generally be composed of two laced channels, preferably rolled ones, although built channels may be used where large sections are required.

Secondary vertical posts may be made of two rolled channels laced, or of four angles in the form of an I with a single line of lacing. These secondary vertical posts should be riveted to the top chords instead of being pin-connected thereto, as in the case of the main vertical posts.

The channels of vertical posts may have their flanges turned either inward or outward, as desired, or so as best to suit the general detailing of the truss.

Stiff bottom chords and inclined web-struts may be made of either two channels with two lines of lacing or of four angles with one line of lacing, the use of trussed eye-bars for struts being prohibited.

Upper lateral struts, overhead transverse struts, and web-stiffening struts shall preferably be made of four angles with one line of lacing. In case, however, the said angles be spaced very far apart, as in lateral struts connecting deep top chords, they are to be placed on the corners of a rectangle, with their legs turned inward, and laced on all four faces of the box strut thus formed.

Eye-bars are to be used for all bottom chords and main diagonals that do not require to be stiffened.

Counters, when employed, can be of either rounds, squares, or flats. These and all other adjustable members are to have their ends enlarged for the screw-threads (unless soft steel, cold-pressed threads be used), so that the diameter at the bottom of the thread shall be one eighth ($\frac{1}{8}$) of an inch greater than that of the body of a round rod of area equal to that of the adjustable piece.

Diagonals for upper lateral systems and vertical sway-bracing shall, preferably, be built of four angles in the form of an I, with a single line of lacing; but for structures where this section would involve an extravagant use of metal, two of the angles, one at the top and one at the bottom, may be omitted, thus making each strut consist of two angles laced, provided, of course, that where the struts cross they shall be rigidly connected by two plates of ample size. This unbalanced section for such diagonals is to be avoided whenever it can be done without undue use of metal. In no case, though, will it be permissible to use angles in tension that are not capable of resisting properly the possible compressive stresses, with due regard for the specified limit of ratio of unsupported length to least radius of gyration.

In cheap highway bridges the lateral diagonals may be made of adjustable rods with right and left clevises at their ends, by which they are to be connected through pins to corner-plates that are riveted to both the lateral strut and the truss member. The ordinary detail consisting of two or three short pieces of angle riveted on top of the cover-plate, and between two of which the rod lies, will not be permitted. Where adjustable rods are employed, the struts to the ends of which they attach must be figured for a total compressive stress equal to the sum of the components (in the direction of said strut) of the greatest allowable working-stresses on all of the adjustable rods meeting at one end of said strut. While this method gives an excessive stress for the strut, the effect will be a desirable error on the side of safety and rigidity.

In designing transverse lateral and overhead struts and their connections, it must be remembered that their main function is to hold rigidly the chords or posts to place and line, and not merely to resist as columns the greatest calculated direct stresses to which they may be subjected. For this reason such struts should have ample section for rigidity, and the connecting plates at their ends should grip both connected members effectively.

Where built stringers are used for the floor system, they shall be made without cover-plates, and generally of the economic depth in respect to total weight of metal, but never less in depth than one fifteenth ($\frac{1}{15}$) of the span. No splices will be allowed in their flanges nor any in their webs, provided that sufficiently long web-plates are procurable. The compression-flanges shall be made of the same gross section as the tension-flanges, and they shall be so stiffened that the unsupported length shall never exceed sixteen (16) times the width of flange. Rigid diagonal bracing of angles is to be used between the top flanges of such stringers, unless they be held rigidly in place by the flooring; and rigid bracing-frames are to be employed between the ends of adjacent stringers at all expansion points. Where such stringers are used, the lower lateral system must invariably consist of rigid sections, each piece being riveted to each stringer where it crosses the same.

In respect to stiffening angles for stringers, the rules governing those for plate-girder spans are to be followed; but the end stiffeners are to be faced or otherwise treated so as to make the stringers of exact length throughout, and so as to effect a uniform bearing of the end stiffeners against the webs of the cross-girders.

In respect to the proportioning of flanges and number of rivets required, the rules given for plate-girder spans are to apply also to stringers. The said rules are to apply to cross-girders, as shall also those relating to stiffeners, splices, cover-plates, and size of compression-flanges, that are given for plate-girder spans. Wherever it is necessary to notch out the corners of the cross-girders to clear the chords, the greatest care must be taken to provide an adequate means for transferring the shear to the posts without impairing either the strength or the rigidity. If necessary, in through-bridges, the web of the cross-girder may be divided into three parts so as to let the end portions project above the top flange and form brackets that will afford opportunity for using an ample number of rivets to connect to the posts, and will strengthen properly the otherwise weakened cross-girder.

All plates, angles, and channels used in built members of trusses must, if practicable, be ordered the full length of the member; otherwise the splices must develop the full strength of the member without any reliance being placed on the abutting ends for carrying compression.

But in total splices at the ends of sections perfect abutting of the dressed ends is to be relied upon. However, the splice-plates even there must be of ample size and strength for both rigidity and continuity.

The unsupported width of plates strained in compression, measuring between centre lines of rivets, shall not exceed thirty-two (32) times their thickness, except in the case of cover-plates for top chords and inclined end posts, where the limit may be increased to forty (40) times the thickness. Where webs are built of two or more thicknesses of plate, the rivets that are used solely for making the several thicknesses act as one plate shall in no case be spaced more than (12) inches from each other, or from other rivets connecting said component thicknesses together. The least allowable thickness for such compound web-plates shall be one (1) inch.

The open sides of all compression-members composed of two rolled or built channels, with or without a cover-plate, shall be stayed by tie-plates at ends and by diagonal lacing-bars or lacing angles at intermediate points. Lacing-bars may be connected to the flanges by either one or two rivets at each end; but lacing angles, which are used for members of heavy section only, must be connected by two rivets at each end.

The tie-plates shall be placed as close as practicable to the ends of the compression-members. Their thickness shall not be less than one fiftieth ($\frac{1}{50}$) of the distance between the centre lines of the rivets by which they are connected to the flanges, unless said tie-plates be well stiffened by angles, in which case they may be made as thin as three eighths ($\frac{3}{8}$) of an inch. The length of a tie-plate shall never be less than its width, or one and one-half ($1\frac{1}{2}$) times the least dimension of strut (unless it be close to a web diaphragm of the member, in which case it may be made as short as twelve (12) inches), and seldom greater than one and one-half ($1\frac{1}{2}$) times its width.

The thicknesses of lacing-bars shall never be less than one fiftieth ($\frac{1}{50}$) of the length between centres of the end rivets, measuring between inmost rivets in case that there be more than one rivet at each end.

The smallest section for a lacing-bar shall be one and three-quarter ($1\frac{3}{4}$) inches by five sixteenths ($\frac{5}{16}$) of an inch, which size may be used for channels under eight (8) inches deep; and the largest section shall be two and a half ($2\frac{1}{2}$) inches by seven-sixteenths ($\frac{7}{16}$) inch, which size shall be used for channels fifteen (15) inches deep. For intermediate sizes of channels, the sizes of lacing-bars shall be interpolated. For all built channels of greater depth than fifteen (15) inches, and for all cases where a lacing-bar would require a greater thickness than seven sixteenths ($\frac{7}{16}$) of an inch, angle lacing is to be used, the smallest section for same being $2'' \times 2\frac{1}{2}'' \times \frac{5}{16}''$, and the largest $2\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$.

In general, the inclination of lacing-bars to axis of member shall be about sixty (60) degrees; but for members of minor importance the said inclination may be made slightly flatter.

Pin-plates shall be used at all pinholes in built members, for the double purpose of reinforcing for the metal cut away and reducing the intensity of pressure on pin and bearing to or below the specified limit. They shall be of such size as to distribute properly, through the rivets, the pressure carried by such plates to both flanges and web of each segment of the member, and shall extend at least six (6) inches within the tie-plates of said member, so as to provide for not less than two (2) transverse rows of rivets there.

When the pin ends of compression-members are cut away into jaw-plates or forked ends, for the purpose of packing closely the various members connected by the pin, these jaw-plates or post extensions shall be considered as columns, the thickness of each of which shall be determined by the following formula:

$$p = 10,000 - 300\frac{l}{t};$$

where p is the greatest allowable intensity of working-stress (impact being considered); l is the unsupported length in inches, measuring from the centre of the pinhole to the centre of the first transverse line of rivets beyond the point at which the full section of the member begins; and t is the total thickness in inches of one jaw. The length l is always to be made as small as practicable, and in cases of unavoidably long extensions the plates are to be stiffened by an interior diaphragm composed of a web with four, or sometimes only two, angles.

It is always better, whenever practicable, to avoid cutting away the ends of channels; but if they must be trimmed, the ends must be reinforced so that the strength of the member shall not be reduced by the trimming.

In riveted tension members the net section through any pinhole shall have an area fifty (50) per cent in excess of the net sectional area of the body of the member. The net section outside of the pinhole along the centre line of stress shall be at least sixty-five (65) per cent of the net section through the pinhole.

Pins are to be proportioned to resist the greatest shearing and bending produced in them by the bars or struts which they connect. No pin is to have a diameter less than eight tenths ($\frac{8}{10}$) of the depth of the deepest eye-bar coupled thereon. No pin is to have a smaller diameter than two and a half ($2\frac{1}{2}$) inches.

Lower chords are to be packed as closely as possible, and in such a manner as to produce the least bending moments on the pins; but adjacent eye-bars in the same panel must never have less than one-half ($\frac{1}{2}$) inch space between them, in order to facilitate painting. The various members attached to any pin must be packed as closely as practicable, and all interior vacant spaces must be filled with steel fillers, where their omission would permit of motion of any member on the pin. All bars are to lie in planes as nearly as possible parallel to the central truss plane.

In detailing I struts composed of four angles with a single line of lacing, the clear distance between backs of angles shall never be made less than three quarters ($\frac{3}{4}$) of an inch, in order to permit the insertion of a small paint-brush.

The greatest allowable pressure upon expansion-rollers of fixed spans, when impact is considered, shall be determined by the equation

$$p = 600d,$$

where p is the permissible pressure in pounds per lineal inch of roller, and d is the diameter of the latter in inches. The least allowable diameter for expansion-rollers is two and a quarter ($2\frac{1}{4}$) inches.

Rollers shall be enclosed in boxes made practically dust-tight, but which will not retain water, and which are so designed that the sides can be readily removed for the purpose of cleaning. These boxes must be so designed as to permit of the free movement of the rollers in the longitudinal direction of span sufficient to take up the extreme variations in length due to temperature changes and deflection, and at the same time prevent any transverse motion of the end of span.

All shoe-plates, bed-plates, and roller-plates are to be so stiffened that the extreme fibre-stress under bending, when impact is included, shall not exceed sixteen thousand (16,000) pounds per square inch.

Pedestals shall be either of cast steel or built up of plates and shapes. In built pedestals, all bearing-surfaces of the base-plates and vertical bearing-plates must be planed. The vertical plates must be secured to the base by angles having at least two rows of rivets in the vertical legs; and the said vertical plates must bear properly from end to end upon said base. No base-plate, vertical plate, or connecting angle shall be less than five-eighths ($\frac{5}{8}$) of an inch in thickness. The vertical plates shall be of sufficient height, and must contain enough metal and rivets to distribute properly the loads over the bearings or rollers. The bases of all cast-steel pedestals shall be planed so as to bear properly on the masonry or rollers.

All rollers and the faces of base-plates in contact therewith are to be planed smooth, so as to furnish perfect contact between rollers and plates throughout their entire length.

Heads of eye-bars are to be made of such dimensions that, when the bars are tested to destruction, they shall break in the body and not in the eyes; and, in the case of loop eyes, so that they shall not fail in the welds. Rods with bent eyes shall not be used. In loop eyes, the distance from the inner point of the loop to the centre of the pinhole must not be less than two and one half ($2\frac{1}{2}$) times the diameter of the pin, and the loop must fit closely to the pin throughout its semi-circumference.

DETAILS OF DESIGN FOR VIADUCTS.

The specifications for the "Details of Design for Trestles and Elevated Railroads" are in general to be followed as far as they will apply in the designing of highway viaducts, the principal variation being that, for cheap structures, adjustable rods with clevises may be substituted for the stiff diagonals in the four faces of the braced towers, by adding, of course, horizontal struts at the panel points of the transverse and longitudinal bracing. These struts must be riveted to the columns by means of wide plates to which the clevises attach, and must never be pin-connected. Corner horizontal plates are to be employed for attaching the horizontal adjustable rods by means of clevises, each of said plates being riveted to both a transverse and a longitudinal bracing strut.

The detailing for the longitudinal girders of viaducts and the bracing between same shall comply with the specifications for detailing highway plate or open-webbed, riveted girder spans; and the specifications for wooden floor system, paving, hand-rails, etc., shall be the same for highway viaducts as for highway bridges.

CHAPTER XVII.

SPECIFICATIONS FOR HIGHWAY DRAW-SPANS.

THESE specifications will be given principally by reference to the previous specifications for Railroad Structures, Highway Bridges, and Railroad Draw-Spans.

GENERAL DESCRIPTION.

CLASSIFICATION.

See Specifications for Highway Bridges.

MATERIALS.

See Specifications for Railroad Draw-spans.

JOISTS, PLANKS, GUARD-TIMBERS, AND WOODEN HAND-RAILS.

See Specifications for Highway Bridges.

FLOORING ON APPROACHES.

See Specifications for Highway Bridges.

STEEL RAILROAD TRACKS.

See Specifications for Highway Bridges.

PAVED FLOORS.

See Specifications for Highway Bridges.

CLEARANCES.

See Specifications for Highway Bridges.

EFFECTIVE LENGTHS AND DEPTHS.

See Specifications for Railroad Structures.

STYLES OF BRIDGES FOR VARIOUS SPAN LENGTHS.

For spans up to one hundred and forty (140) feet in length, plate-girder spans are to be used. These plate-girder spans may be made to act as continuous girders over the pivot-pier, or may have pin-connections over the drum, so that when the live load is applied they will act as two separate spans. The former style is generally preferable as a matter of economy in time of operation, there being no important reason for raising the ends to any great extent, as there is in the case of railroad draw-spans.

For spans between one hundred and forty (140) and two hundred and twenty-five (225) feet, pin-connected Pratt trusses with parallel chords are to be used.

For spans between two hundred and twenty-five (225) feet and three hundred (300) feet, pin-connected Pratt trusses with broken top chords are to be employed.

For spans of over three hundred (300) feet, pin-connected trusses with subdivided panels are to be adopted.

It is understood that these limiting lengths are not fixed absolutely, as the best limits will vary somewhat with the width of bridge and the live load to be carried.

The proper truss depths for all cases cannot well be specified, as they will depend upon various considerations, such as appearance, economy, width of structure, etc.

In all cases the top chords are to be of rigid members, and inclined posts are to be used at ends and over drum, as specified for railroad draw-spans.

MAIN MEMBERS OF TRUSS DRAW-SPANS.

See Specifications for Highway Bridges.

LOADS.

See Specifications for Railroad Draw-Spans.

LIVE LOADS.

See Specifications for Highway Bridges; and, for the manner of applying live loads to draw-spans, see Specifications for Railroad Draw-Spans.

IMPACT-ALLOWANCE LOAD.

See Specifications for Highway Bridges.

DEAD LOAD.

See Specifications for Railroad Structures.

ASSUMED UPLIFT STRESSES.

See Specifications for Railroad Draw-Spans.

The inferior limit of uplift for designing the machinery of light highway drawbridges is to be taken at ten thousand (10,000) pounds at each of the four corners of the span.

WIND LOADS.

See Specifications for Highway Bridges. For method of using the wind loads, see Specifications for Railroad Draw-Spans.

INDIRECT WIND LOAD OR TRANSFERRED LOAD.

See Specifications for Railroad Structures. For method of dealing with this load, see Specifications for Railroad Draw-Spans.

INTENSITIES OF WORKING-STRESSES.

See Specifications for Railroad Structures.

BEARINGS UPON MASONRY.

See Specifications for Railroad Structures.

REVERSING STRESSES.

See Specifications for Railroad Structures.

NET SECTION.

See Specifications for Railroad Structures.

BENDING MOMENTS ON PINS.

See Specifications for Railroad Structures.

COMBINATIONS OF STRESSES.

See Specifications for Railroad Draw-Spans.

It is to be observed, however, that, for spans which do not carry trains, the live load and the wind load are assumed not to act simultaneously.

BENDING ON TOP CHORDS.

See Specifications for Railroad Structures.

BENDING ON INCLINED END POSTS.

The Specifications for Railroad Structures under this heading are to be followed with this exception: in bridges that do not carry trains, the live load and the wind load are assumed not to act simultaneously.

BENDING DUE TO WEIGHT OF MEMBER.

See Specifications for Railroad Structures.

GENERAL LIMITS IN DESIGNING.

See Specifications for Highway Bridges.

GENERAL PRINCIPLES IN DESIGNING.

See Specifications for Railroad Structures.

RIVETING.

See Specifications for Highway Bridges.

DETAILS OF DESIGN FOR PLATE-GIRDER DRAW-SPANS.

The specifications for the corresponding item in the Specifications for Railroad Draw-Spans are to be followed, with the following exceptions:

1st. The perpendicular distances between central planes of girders will be made to suit the general requirements; and, 2d. At least eight (8) points of support on the drum will be needed.

DETAILS OF DESIGN FOR PIN-CONNECTED DRAW-SPANS.

The specifications for the corresponding item in the Specifications for Highway Bridges are to be followed, and in addition thereto those given under the heading "Details of Design for Trusses of Draw-Spans" in the Specifications for Railroad Draw-Spans are to be employed, except that the use of adjustable members for lateral diagonals will be permitted in the case of cheap highway draw-spans.

DETAILS OF DRUMS AND TURNTABLES.

In general the Specifications for the corresponding item in the Specifications for Railroad Draw-Spans shall be followed, except that, for light, highway draws, the limiting thicknesses, etc., may be reduced to the following:

Top flanges and webs of drums—three eighths ($\frac{3}{8}$) of an inch.

Bottom flanges of drums—five eighths ($\frac{5}{8}$) of an inch.

Upper track segments—one and three-quarter ($1\frac{3}{4}$) inches.

Lower track segments—two (2) inches.

Bearing-plates over drum—three quarters ($\frac{3}{4}$) of an inch.

Centre casting on pivot-pier—one (1) inch.

Anchor-bolts for same—one and one-eighth ($1\frac{1}{8}$) inches in diameter and two and a half ($2\frac{1}{2}$) feet long.

Rollers—ten (10) inches in diameter and six (6) inches face.

MACHINERY FOR TURNING THE SPAN AND LIFTING THE ENDS OF SAME.

See Specifications for Railroad Draw-Spans.

METHOD OF DETERMINING POWER REQUIRED FOR OPERATING THE SPAN AND LIFTING THE ENDS.

See Specifications for Railroad Draw-Spans.

DETAILS OF MACHINERY.

OPERATING MACHINERY.

See Specifications for Railroad Draw-Spans.

END-LIFTING APPARATUS.

See Specifications for Railroad Draw Spans.

SHOES AND END-BEARING ROLLERS.

See Specifications for Railroad Draw-Spans.

HOUSES AND SUPPORTS.

See Specifications for Railroad Draw-Spans.

CAMBER AND DEFLECTION.

See Specifications for Railroad Draw-Spans.

CHAPTER XVIII.

GENERAL SPECIFICATIONS GOVERNING THE MANUFACTURE, SHIPMENT, AND ERECTION OF STEEL BRIDGES, TRESTLES, VIADUCTS, AND ELEVATED RAILROADS.

DRAWINGS.

As soon as practicable after the signing of the contract for building the structure, complete detail drawings will be furnished by the Engineer, and from these the Contractor is to prepare his shop drawings, complying carefully therewith and making no changes without the written consent of the Engineer. The working drawings are to be sent in triplicate for the approval of the Engineer and his principal Shop Inspector, who will retain two sets and return the third after checking same and marking thereon any changes or corrections desired; after which a corrected set of shop drawings shall be sent without delay by the Contractor to the Engineer. The approval of said working drawings by the Engineer will not relieve the Contractor from the responsibility of any errors thereon.

The drawings furnished by the Engineer shall be checked carefully by the Contractor before beginning work. Should any errors be discovered, the Engineer's attention shall be called to same, and corrections will be made, after which the Contractor shall be responsible for all errors which may occur or which may have occurred. The Engineer shall have the right to alter as he may see fit the preliminary plans, if further investigation of the conditions affecting the proposed structure so warrant; and he shall be at liberty to make minor changes in all plans during construction without any extra charge for same being made by the Contractor unless, in the opinion of the Engineer, the Contractor be really entitled to extra compensation on account of such changes.

The Contractor shall furnish without charge as many sets of shop drawings as the Engineer and other officers of the company may deem necessary for their use during construction or for record.

INSPECTION.

The inspection and tests of metal will be made promptly on its being rolled or cast, and the quality will be determined before it leaves the rolling-mill or foundry. The inspection of workmanship will be made as the manufacture of the material progresses, and at as early a period as the nature of the work will permit.

All facilities for inspection of material and workmanship shall be furnished by the Contractor; and the Engineer and his inspectors shall have free access to any of the works in which any portion of the material is being made.

The Contractor shall give the Inspector due notice when any material is ready for inspection. Any delay on the part of the Inspector shall be reported to the Engineer, but no material will be accepted which has not been passed upon by the authorized representative of the Engineer.

METAL.

Unless otherwise specified, all metal shall, for all spans of ordinary length, be medium steel; excepting only that rivets and bolts are to be of soft steel, and adjustable members of either soft steel or wrought iron.

For very long, fixed spans, high steel may be used for top chords, inclined end posts, pins, and eye-bars in bottom chords and in main diagonals of panels where there is no reversion of stress, when impact is included. It may be used also for the web members of cantilever and anchor arms of cantilever bridges, where the variation of stress is comparatively small, and where the impact cannot be great.

Except for purely ornamental work and a few minor details of the operating machinery of drawbridges, cast iron will not be allowed to be used in the superstructure of any bridge, trestle, viaduct, or elevated railroad, cast steel being employed wherever important castings are necessary.

ROLLED STEEL.

All soft and medium steel shall be manufactured by either the acid or the basic open-hearth process, but high steel shall invariably be manufactured by the former. All steel must be uniform in character for each specified kind. Any attempt to substitute Bessemer or any other steel for the open-hearth product will be considered as a violation of the contract and a good and sufficient reason for cancelling the same.

All plates shall be rolled from slabs. These slabs shall be made by a separate operation, by rolling an ingot and cutting off the scrap. The original ingot shall have at least twice the cross-sectional area of the slab, and the latter shall be at least six times as thick as the plate.

All finished material coming from the mills must be free from seams, flaws, or cracks, and must have a clean, smooth finish.

COMPOSITION OF ROLLED STEEL.

The greatest allowable percentages of certain principal ingredients of the various kinds of rolled steel shall be as given in the following table:

Ingredients.	Percentages.		
	Soft Steel.	Medium Steel.	High Steel.
Phosphorus (acid steel)..	0.05	0.06	0.07
Phosphorus (basic steel).	0.03	0.04	....
Sulphur....	0.04	0.05	0.05
Silicon................ ...	0.04	0.05	0.06
Manganese...	0.60	0.70	0.80

These percentages apply to drillings taken from the edges of plates and the exterior of shapes, bars, or flats. If, however, the drillings be taken from the middle of plates or the heart of other sections, the percentages given in the table are to be increased twenty-five (25) per cent.

IDENTIFICATION.

Each ingot shall be stamped or marked plainly with its proper melt-number; and this melt-number must be stamped or painted plainly on all blooms, billets, or slabs made from such ingots in order to identify the material throughout its various processes of manufacture; and the melt-number must be stamped plainly on each piece of finished material. Rivet and lacing steel, and small pieces for pin-plates and stiffeners, may be shipped in bundles, securely wired together, with the blow or melt number on a metal tag attached.

GENERAL PROVISIONS ON METHODS OF TESTING.

Rivet-rods and other rounds are to be tested in the form in which they leave the rolls, without machining.

Test-pieces from angles, plates, shapes, etc., shall be rectangular in shape, with a cross-sectional area of preferably about one half (½) of a square inch, but not less, and shall be taken so that only two sides are machine-finished, the other two having the surface which was left by the rolls.

Should fracture occur outside of the middle third of the gauge length, the test is to be discarded as worthless if it falls below the standard.

If any test-piece have a manifest flaw, its test shall not be considered.

In case that one test-piece falls slightly below the requirements in any particular, the Inspector may allow the re-testing of the lot or heat by taking four (4) additional tests from the said lot or heat; and, if the average of the five (5) shall show that the steel is within the requirements, the metal may be accepted; otherwise it shall be rejected.

Drillings for chemical analysis may be taken either from the preliminary test-piece or from the finished material.

The speed of the machine for breaking test-pieces shall not be less than one-quarter (¼) inch per minute, nor more than three (3) inches per minute.

Material which is to be used without annealing or further treatment is to be tested in the condition in which it comes from the rolls. When the material is to be annealed or otherwise treated before use, the specimens representing such material may be similarly treated before testing; but they shall also give standard elongation, reduction, and fracture before annealing.

TENSILE STRENGTH.

The ultimate tensile strength per square inch on test-pieces for all three kinds of rolled steel used in structural metal-work shall be as follows:

Soft steel	50,000 lbs. to 60,000 lbs.
Medium steel ...	60,000 lbs. to 70,000 lbs.
High steel	70,000 lbs. to 80,000 lbs.

ELASTIC LIMITS.

The least allowable elastic limits obtained from test-pieces and determined in the usual manner by the drop of the beam shall be as follows:

Soft steel	30,000 lbs. per square inch.
Medium steel...	35,000 lbs. per square inch.
High steel	40,000 lbs. per square inch.

ELONGATION.

The percentages of elongation shall be obtained from the test-pieces after breaking on an original length of eight (8) inches, in which length must occur the curve of reduction from stretch on both sides of the point of fracture. The least allowable elongations for the various kinds of rolled structural steel shall be as follows,

Shape.	Percentage of Elongation.		
	Soft Steel.	Medium Steel.	High Steel.
Rounds (excepting pins)	29	27	—
Pins	—	20	18
Angles and bars	29	26	22
Plates under 40″ wide	—	24	20
Plates 40″ to 70″ wide, and webs of beams and channels	—	23	19
Plates over 70″ wide	—	21	—
Flanges of beams and channels	—	20	18

REDUCTION OF AREA.

The reduction of area, measured on test-pieces, for the various kinds of rolled structural steel shall be as follows:

Shape.	Percentage of Reduction of Area.		
	Soft Steel.	Medium Steel.	High Steel.
Rounds (excepting pins)	50	44	38
Pins	—	40	34
Angles and bars	48	40	34
Plates under 40″ wide	—	40	34
Plates 40″ to 70″ wide, and webs of beams and channels	—	38	34
Plates over 70″ wide	—	37	—
Flanges of beams and channels	—	36	30

BENDING TESTS.

Specimens of soft steel shall be capable of bending to one hundred and eighty (180) degrees and closing down flat upon themselves, without cracking, when either hot, cold, or quenched.

Specimens of medium steel, when heated to a dark orange and cooled in water at seventy (70) degrees Fahrenheit, or when cold or hot, shall be capable of bending one hundred and eighty (180) degrees around a circle whose diameter is equal to the thickness of the test-piece, without showing signs of cracking on the convex side of the bend.

Specimens of high steel when quenched in a similar manner shall be capable of bending ninety (90) degrees around a circle whose diameter is equal to twice the thickness of the test-piece, and one hundred and eighty (180) degrees, either hot or cold, without showing signs of cracking on the convex side of the bend.

DRIFTING TESTS.

Punched rivet-holes in medium steel, pitched two (2) diameters from a sheared edge, must stand drifting until their diameters are fifty (50) per cent greater than those of the original holes, and must show no signs of cracking the metal.

High steel must stand the same test, except that the increase in diameter is to be twenty-five (25) per cent instead of fifty (50) per cent.

FRACTURE.

All broken test-pieces for all three classes of steel must show a silky fracture of uniform color.

NUMBER OF TEST-PIECES.

At least three (3) tensile tests and three bending tests shall be made on specimens from different ingots of each melt. The bending tests may, if desired, be made on the broken test-pieces of the tension tests. If material of various shapes is to be made from the same melt, the specimens for testing are to be so selected as to represent the different shapes rolled from such melt.

All tests are to be made by the Contractor for the Inspector without charge.

The Inspector will be permitted considerable latitude in respect to the number of tests required, reducing same when the metal runs uniformly and increasing same when it does not.

Lots for testing shall not exceed twenty (20) tons in weight; and plates rolled in universal mill or in grooves, or sheared plates, shall each constitute a separate lot, as shall also the angles, channels, or beams.

TESTS OF FULL-SIZED EYE-BARS.

Full-sized eye-bars may be tested to destruction, provided notice be given in advance of the number and size required for this purpose, so that the material can be rolled at the same time as that required for the structure. The number of tests of full-sized eye-bars will depend upon the size of the order and upon the regularity of the results of the tests. In general, for small orders, the number of tests shall be about three (3) per cent of the number of eye-bars in the order, but never less than two bars for an order for a single span. For large orders the number of tests shall be about two (2) per cent of the number of eye-bars in the order. Should the Inspector find the bars to be very uniform in strength, elasticity, and ductility, and fully up to the specifications, he will be at liberty to reduce the number of tests of full-sized bars. In the case of testing long bars, it will be allowable to choose a bar at random from a number of finished bars, cut it in two, and upset the cut end of each piece, thus making two test-bars.

Full-sized bars of medium steel must show an ultimate tensile strength of at least sixty thousand (60,000) pounds per square inch for bars of one (1) inch thickness and under, and not less than fifty-six thousand (56,000) pounds per square inch for bars of two (2) inches thickness and over. Bars with thicknesses between these limits must show proportionate strength. The elongation shall be not less than fourteen (14) per cent in a gauged length of ten (10) feet; and the elastic limit shall not be less than fifty-five (55) per cent of the ultimate strength of the bar for bars not over one inch thick, or less than fifty (50) per cent of same for bars of two (2) inches thickness and over, with proportionate percentages for bars of intermediate thicknesses.

For high steel the limits just specified shall be changed as follows:

Ultimate strength, 70,000 to 65,000 pounds.

Elongation, twelve (12) per cent.

Elastic limit, fifty-two (52) per cent to forty-seven (47) per cent.

Any lot of steel bars which meets the requirements of the preceding paragraph shall be accepted, if none of the bars which break in the eye show an ultimate strength, elastic limit, or elongation less than that specified for the body of the bar, unless one fourth (¼) of the full-sized samples so tested break in the eye. In case of failure to meet any of these requirements, the lot from which the sample bars were taken will be rejected.

All full-sized sample bars which break at less than the ultimate strength specified, or do not otherwise fill the specifications, shall be at the expense of the Contractor; unless, in case of those that break in the eye, he shall have made objection in writing to the form or dimensions of the heads before making the eye-bars. All others shall be paid for by the purchaser at the contract price of finished metal-work on cars at shops, less the scrap value of the broken bars.

PIN METAL.

Pins up to six (6) inches in diameter may be rolled, but above that diameter they shall be forged. The rounds from which the pins are to be turned must be true, straight, and free from all injurious flaws or cracks. All forged pins shall be reduced from a single bloom or ingot until perfect homogeneity is secured throughout the whole mass. The blooms shall have at least three (3) times the sectional area of the finished pins. No forging shall be done below a red heat.

VARIATION IN WEIGHT.

Except in the case of sheared plates ordered to gauge, a variation in cross-section or weight of rolled material of more than two (2) per cent from that specified may be cause for rejection. For the said sheared plates the permissible excess variation shall run from four (4) per cent for plates five eighths (⅝) of an inch or more in thickness to eight (8) per cent for plates three eighths (⅜) of an inch or less in thickness, the variations for intermediate thicknesses being directly interpolated.

Should the shipping weight of any entire order exceed by more than one (1) per cent the weight computed from the approved shop drawings, the amount in excess of the said one (1) per cent will not be paid for, unless in the entire order the weight of plates exceeding thirty-six (36) inches wide be greater than thirty (30) per cent of the whole, in which case the allowable variation shall be increased to two (2) per cent.

WROUGHT IRON.

All wrought iron, if any be used, must be of the best quality obtainable, tough, ductile, fibrous, and of a uniform quality; also straight, smooth, and free from cinder-pockets or injurious flaws, buckles, blisters, or cracks. No steel scrap shall be used in its manufacture.

The tensile strength, determined from test-pieces in the same manner as specified for steel, shall not fall below fifty thousand (50,000) pounds per square inch; and the elastic limit shall not be less than twenty-six thousand (26,000) pounds per square inch. The elongation, determined in the same manner as specified for steel, shall not be less than twenty (20) per cent.

All wrought iron must bend cold one hundred and eighty (180) degrees, without sign of fracture, to a curve the inner radius of which equals the thickness of the piece tested. Soft steel is to be used instead of wrought iron wherever practicable.

CAST IRON.

Except where chilled iron is specified, all castings shall be of tough, gray iron, free from injurious cold-shuts or blow-holes, true to pattern, and of a workmanlike finish. Sample-pieces one (1) inch square, cast from the same heat of metal in sand-moulds, shall be capable of maintaining on a clear span of four (4) feet six (6) inches a central load of five hundred (500) pounds when tested in the rough bar. All castings shall be straight and out of wind, with proper and approved uniform thickness of metal, and shall have perfect, sharp, and clean lines, angles, and mouldings, all re-entrant angles being properly filleted.

CAST STEEL.

All steel castings shall be made of acid open-hearth steel containing from twenty-five hundredths (0.25) to four tenths (0.4) per cent of carbon, and not more than the following percentages of other ingredients:

Phosphorus, five hundredths (0.05).
Sulphur, five hundredths (0.05).
Manganese, eight tenths (0.8).

The ultimate tensile strength shall run from sixty-five thousand (65,000) to seventy-five thousand (75,000) pounds per square inch; the elastic limit shall not be less than one half (½) of the ultimate strength; and the elongation of test specimens in two (2) inches shall not be less than fifteen (15) per cent for fixed castings or seventeen (17) per cent for movable castings.

All steel castings shall be carefully and uniformly annealed, and shall be true to drawings, smooth, clean, and free from blowholes, sponginess, and all other defects. All corners therein shall be properly filleted.

TESTS OF ROLLERS FOR DRAW-SPANS.

The Contractor shall make, at his own expense, under the direction of the Engineer or his duly authorized representative, for each draw-span, tests, not exceeding three (3) in number, of full-sized cast rollers; also any tests of specimens of the metal for the same that may be considered necessary by the Engineer to determine its quality.

OTHER TESTS OF FULL-SIZE MEMBERS OR DETAILS.

The Contractor shall make, at his own expense, under the direction of the Engineer or his Inspector, such other tests of full-size members or details as the Engineer may prescribe, provided that the said members or details are similar to those used on the work, and provided that the total cost to the Contractor of such extra tests does not exceed one quarter of one per cent (0.25%) of the total contract price of the work.

WORKMANSHIP.

All metal shall be carefully straightened before being turned over to the shops.

All workmanship shall be first-class in every particular, and all portions of metal-work exposed to view shall be neatly finished.

All idle corners of plates and angles, such for instance as the ends of the unconnected legs of angle lacing, shall be neatly chamfered off at an angle of about forty-five (45) degrees, so as to give a sightly finish to the work and to avoid bending of said corners during shipment and erection.

As far as practicable, all parts shall be so constructed as to be accessible for inspection and painting.

All punched work shall be so accurately done that, after the various component pieces are assembled and before the reaming is commenced, forty (40) per cent of the holes can be entered easily by a rod of a diameter one sixteenth ($\frac{1}{16}$) of an inch less than that of the punched holes; eighty (80) per cent by a rod of a diameter one eighth ($\frac{1}{8}$) of an inch less than same; and one hundred (100) per cent by a rod of a diameter one quarter ($\frac{1}{4}$) of an inch less than same. Any shopwork not coming up to this requirement will be subject to rejection by the inspector.

SHEARED EDGES.

All sheared and hot-cut edges shall have not less than one quarter ($\frac{1}{4}$) inch of metal removed by planing to a smooth, finished surface. Lacing-bars, fillers, stay-plates, and stringer-bracing connecting plates only will be exempt from this requirement.

RE-ENTRANT CORNERS.

No sharp or unfilleted re-entrant corners will be allowed anywhere in the work.

ANNEALING.

In all cases where a steel piece in which the full strength is required has been partially heated or bent, the whole piece must be subsequently annealed. In pieces of secondary importance, where the bending is slight, said bending is to be made cold, and no annealing in such cases will be required. Crimped web-stiffeners will not require annealing.

All eye bars shall be carefully and uniformly annealed at a dark orange heat.

RIVETS.

Rivets when driven must completely fill the holes, have full heads concentric with the rivet-holes, and be machine-driven whenever practicable. The machine must be capable of retaining the applied pressure after the upsetting is completed.

The rivet-heads must be full and neatly finished, of approved hemispherical shape, in full contact with the surface, or be countersunk when so required, and of a uniform size for the same-sized rivets throughout the work; and they must pinch the connected pieces thoroughly together. Flattened heads may be used in certain places, if necessary for clearance. Except where shown otherwise on the drawings, all rivet diameters are to be seven eighths ($\frac{7}{8}$) of an inch. No loose or imperfect rivets will be allowed to remain in any part of the metal-work.

RIVET-HOLES.

Rivet-holes must be accurately spaced; the use of drift-pins will be allowed only for bringing together the several parts forming a member, and they must not be driven with such force as to distort the metal about the holes. The distance between the edge of any piece and the centre of a rivet-hole must never be less than one and a half ($1\frac{1}{2}$) inches, excepting for lattice bars, small angles, and where especially shown otherwise on the Engineer's drawings; and, wherever practicable, this distance shall be at least two (2) diameters of the rivet.

PUNCHING AND REAMING.

All rivet-holes in steel-work, if punched, shall be made with a punch one-eighth ($\frac{1}{8}$) inch in diameter less than the diameter of the rivet intended to be used, and shall be reamed to a diameter one-sixteenth ($\frac{1}{16}$) inch greater than that of the said rivet.

Before this reaming takes place all the pieces to be riveted together shall be assembled and bolted into position, then the reaming shall be done; for one of the principal objects of this clause in relation to subpunching is to ensure the correct matching of rivet-holes, and the avoidance of holes of excessive diameter. Said clause also ensures the removal of most, if not all, incipient cracks started by the process of punching.

All reaming is to be done by means of twist-drills, the use of tapered reamers being prohibited, except where twist-reamers cannot be employed. All holes must be at right angles to surface of member, and all sharp or raised edges of holes under heads must be slightly rounded off before the rivets are driven.

All holes for field-rivets, excepting those for lateral and sway-bracing, when not drilled to an iron template, shall be reamed while the connecting parts are temporarily assembled.

Punching shall not be permitted in any piece in which the thickness of the metal exceeds the diameter of the cold rivet that is to be used; but all such pieces shall be drilled solid.

BUILT MEMBERS.

Built members must, when finished, be true and free from twists, kinks, buckles, or open joints between the component pieces.

All abutting surfaces of compression-members, except flanges of plate-girders where the joints are fully spliced, must be planed or turned to even bearings so that they shall be in as perfect contact throughout as can be obtained by such means; and all such finished surfaces must be protected by white lead and tallow before shipment from the shop.

The ends of all webs and chord or flange angles that abut against other webs must be faced true and square or to exact bevel; and the end-stiffeners must be placed perfectly flush with these planed ends, so as to afford a proper bearing. Filling-plates beneath end-stiffening angles must be practically flush with said angles, and must in no case project outside of same at the bearings. If a good and satisfactory job of work cannot be obtained by this method, the end-stiffening angles shall be made one eighth ($\frac{1}{8}$) of an inch thicker, and the entire ends shall be planed after the stiffening angles are riveted on.

No web-plate will be allowed to project beyond the flange angles, or to recede more than one eighth ($\frac{1}{8}$) of an inch from faces of same.

All filling and splice plates in riveted work must fit at their ends to the flanges sufficiently close to be sealed, when painted, against the admission of water; but they need not be tool-finished, unless so specially indicated either on the drawings or in the specifications.

EYE-BARS.

Except in the case of loop-eyes, no weld will be allowed in the body of the eye-bars. The heads of the eye-bars shall be made by upsetting, rolling, or forging into shape. A variation from the specified dimensions of the heads will be allowed, in thickness of one thirty-second ($\frac{1}{32}$) of an inch below and one sixteenth ($\frac{1}{16}$) of an inch above that specified, and in diameter of one fourth ($\frac{1}{4}$) of an inch in either direction. Eye-bars must be perfectly straight before boring.

Loop-eyes shall invariably be made of wrought iron, as steel cannot be relied upon to afford a proper weld.

PINHOLES.

All pinholes must be bored truly parallel and at right angles to the axes of the members, unless otherwise shown on the drawings; and, in pieces not adjustable for length, no variation of more than one thirty-second ($\frac{1}{32}$) of an inch will be allowed in the length between centres of pinholes.

Pinholes in eye-bars must be in the centre of the heads, and on the centre line of the bars.

Bars, which are to be placed side by side in the structure, shall be bored at the same temperature, and shall be of such equal length that, when placed in a pile, the pin at each end will pass through the holes at the same time without forcing.

PINS.

All pins shall be turned accurately to a gauge, and shall be finished perfectly round, smooth, and straight. All pins up to three and one-half ($3\frac{1}{2}$) inches in diameter shall fit the pin-holes within one fiftieth ($\frac{1}{50}$) of an inch, and all pins over three and one-half ($3\frac{1}{2}$) inches in diameter shall fit their holes within one thirty-second ($\frac{1}{32}$) of an inch.

The Contractor must provide steel pilot-nuts for all pins to preserve the threads while said pins are being driven.

TURNED BOLTS.

When members are connected by bolts which transmit shearing-stresses, the holes must be reamed parallel, and the bolts must be turned to a driving fit.

TURNBUCKLES, NUTS, THREADS, AND WASHERS.

All sleeve-nuts, turnbuckles, and clevises must be made so strong and stiff that they will be able to resist without rupture the ultimate pull of the bars which they connect, and without distortion the greatest twisting force to which they could ever be subjected. They must be made so that the threaded lengths of the rods engaged can be verified.

The dimensions of all square and hexagonal nuts, except those on the ends of pins, shall be such as to develop the full strength of the body of the adjustable member. No round-headed bolts will be allowed.

Washers and nuts must have uniform bearing.

Cast or wrought iron washers must be used under the heads of all timber bolts when the bearing is on the wood.

All threads, except those on the ends of pins, must be of the United States standard. Each adjusting nut must be provided with an effective nut-lock or check-washer.

ROLLERS.

Rollers shall be turned accurately to a gauge, and must be finished perfectly round, and to the correct diameter or diameters, from end to end. The tongues and grooves in plates and rollers must fit snugly, so as to prevent lateral motion. Roller-beds must be planed.

ANCHOR-BOLTS.

All bed-plates and bearings must be fox-bolted to the masonry or attached to concrete by anchor-plates. The Contractor must furnish all bolts, drill all holes, and set the bolts to place with Portland-cement grouting.

All anchor-bolts are to be of soft steel with cold-pressed threads; and the threaded portion of all such bolts tested to destruction shall develop a greater strength than that of the unthreaded portion of same. The lengths of the nuts for all adjustable rods must always be great enough to develop the full strength of the rod.

All anchor-bolts are to be thoroughly oiled but not painted before shipment; and the exposed portions thereof, after erection, are to receive two coats of paint at the same time the rest of the metal-work receives its two coats.

NAME PLATES.

A name-plate of neat design and finish, giving the name of the Contractor and the date of erection, shall be firmly attached to each end of every through-bridge, and to some prominent place or places in all other structures.

PAINTING.

All metal-work before leaving the shop shall be thoroughly cleansed from all loose scale, rust, and dirt, and shall then be given one coat of the best carbon primer, Eureka paint, or any other priming-coat required by the Engineer, which coat shall be thoroughly dried before the metal-work is loaded for shipment. It is absolutely essential that the entire surface of the metal-work be thoroughly cleansed by the most effective known methods, such as the use of wire-brushes, then the painter's torch, and in certain cases the application of a strong caustic solution, followed by scraping, washing with clean water, and drying.

In riveted work all surfaces coming in contact shall be extra well painted before being riveted together. Bottoms of bedplates, bearing-plates, and any other parts which are not accessible for painting after erection shall have three coats of paint, one at the shop, the other two in the field, before erection. Pins, bored pinholes, turned friction-rollers, and all other polished surfaces shall be coated with white lead and tallow before shipment from the shop.

After the structure is erected the metal-work shall be thoroughly cleansed from mud, grease, or any other objectionable material that may be found thereon, then thoroughly and evenly painted with two (2) coats of paint of any kind that the Engineer may adopt.

All three coats of paint given to the metal-work are to be of distinctly different shades or colors; and the second coat must be allowed to dry thoroughly before the third coat is applied.

No thinning of paint with turpentine, benzine, or other thinner will be allowed without special written permission from the Engineer.

No painting is to be done in wet or freezing weather.

All painting is to be done in a thorough and workmanlike manner, to the satisfaction of the Engineer, and no paint whatever is to be used on the structure without first being approved by the Engineer.

All the materials for painting shall be subject at all times to the closest inspection and chemical analysis; and the detection of any inferior quality of such material, in either shop or field, shall involve the rejection of all such suspected material at hand and the scraping and repainting of those portions of the work which, in the opinion of the Engineer, were defectively painted on account of such inferior material.

All recesses which would retain water or through which water could enter must be filled with thick paint or some water-proof cement before receiving final painting. All surfaces so close together as to prevent the insertion of paint-brushes must be painted thoroughly by using a piece of cloth instead of the brush.

SHIPPING.

All parts shall be loaded carefully so as to avoid injury in transportation, and shall be at the Contractor's risk until erected and accepted.

In shipping long plate girders great care is to be taken to distribute the weight properly over the two cars that support them, and to provide means for permitting the cars to pass around curves without disturbing the loading. In both the handling and shipment of metal-work every care is to be taken to avoid bending or straining the pieces or damaging the paint. All pieces bent or otherwise injured will be rejected.

TIMBER.

All timber must be of the best quality, sawed true and out of wind, full size, and free from wind-shakes, large or loose knots, decayed wood, sap, worm-holes, or any other defect that would impair its strength or durability.

ERECTION.

The Contractor shall furnish all staging and falsework, and shall erect, adjust, and paint all of the metal-work ready for the timber-floor. He shall also furnish and lay the latter and put on the track-rails, unless there be a written agreement to the contrary.

The Contractor shall employ suitable mechanics for every kind of mechanical work, and shall, at the request of the engineer, discharge any workman whom the said Engineer shall deem incompetent, negligent, or untrustworthy.

All material of whatever kind shall be subject to inspection and approval at any time during the progress and until the final completion of the work; and the entire work shall be constructed in a substantial and workmanlike manner, and to the satisfaction and acceptance of the Engineer.

DEFECTIVE WORK.

The Contractor, upon being so directed by the Engineer, shall remove, rebuild, or make good, without charge, any work which the said Engineer may consider to be defectively executed. The fact that any defective material in the structure had been previously accepted by the oversight of the Company's engineers or inspectors shall not be considered a valid reason for the Contractor's refusing to remove it or make it good. And until such defective work is removed and made good, the Engineer shall deduct from the partial payments or the final payment, as the case may be, whatever sum for such defective work as may, in his opinion, appear just and equitable.

DIRECTIONS TO CONTRACTOR.

In case that the Contractor shall not be present upon the work at any time when it may be necessary for the Engineer to give instructions, the foreman in charge for the time being shall receive and obey any orders that the Engineer may give.

The Contractor shall commence work at such points as the Engineer may direct, and shall conform to his directions as to the order and time in which the different parts of the work shall be done, as well as to the force required to complete the work at the date specified.

CLOSING THOROUGHFARES.

The Contractor and his employees shall so conduct their operations as not to close any thoroughfare by land or water without the written consent of the proper authorities of such thoroughfare.

RESPONSIBILITY FOR ACCIDENTS.

The Contractor shall assume and be responsible for all accidents to men, animals, and materials before the acceptance of the structure; and must remove at his own expense all false-work, rubbish, or other useless material caused by his operations; and such work shall be included as a part of the work to be performed.

The Contractor shall place sufficient and proper guards for the prevention of accidents, and shall put up and maintain at night suitable and sufficient lights.

DAMAGES.

The Contractor shall indemnify and save harmless the Company against all claims and demands of all parties whatsoever for damages or compensation for injuries arising from any obstructions created by the Contractor or his employees, or from any neglect or omission to provide proper lights and signals during the construction of the work.

ALTERATION OF PLANS.

The Engineer shall have the power to vary, extend, increase, or diminish the quantity of the work, or to dispense with a portion thereof during its progress without impairing the contract; and no allowance will be made the Contractor except for the work actually done. In case any change involve the execution of work of a class not herein provided for, the Contractor shall perform the same and be paid the actual cost thereof plus the percentage for profit agreed upon in the contract. In this case the Contractor must furnish the Engineer with satisfactory vouchers for all labor and material expended on the work.

STRICTNESS OF INSPECTION.

All materials and workmanship will be thoroughly and carefully inspected, and the Contractor will be held at all times to the spirit of the specifications; but nothing will be done by the Company's engineers or inspectors to give the Contractor needless worry or annoyance, the intent of both specifications and inspection being simply to obtain for the Company work that will be first-class in every particular and a credit to every one connected with its designing and construction.

SPIRIT OF THE SPECIFICATIONS.

The nature and spirit of these specifications are to provide for the work herein enumerated to be fully completed in every detail for the purpose designed; and it is hereby understood that the Contractor, in accepting the contract, agrees to furnish any and every thing necessary for such construction, notwithstanding any omission in the drawings or specifications.

ENGINEER.

Whenever in these specifications the term "Engineer" is employed, it is understood that it is to mean the ... . Engineer of the .... Company or the duly authorized representative of same.

TENDERS.

Tenders for all work, whenever it is practicable, shall be made on schedule prices, lump-sum bids being accepted only for such parts as steam or electric machinery, which could not well be paid for by the pound.

All tenders are to be made in strict accordance with the plans and specifications submitted to bidders by the Engineer; and no bids based upon suggested changes in same will be considered.

In awarding contracts, preference will be given to those bidders in whose shops the piece-work system is least employed.

CHAPTER XIX.

THE COMPROMISE STANDARD SYSTEM OF LIVE LOADS FOR RAILWAY BRIDGES AND THE EQUIVALENTS FOR SAME.

IN 1893 the author published a small pamphlet, now out of print, which bore the above title. Its contents are reproduced here instead of in a second edition. The various steps taken in its preparation were as follows:

In 1891 the author presented to the American Society of Civil Engineers a paper entitled "Some Disputed Points in Railway-Bridge Designing," in which he advocated the adoption of a few standard train-loads for railroad bridges, instead of the almost innumerable ones then in use, offered a set of loads for discussion, and urged that the "Equivalent Uniform-Load Method" of computing stresses be adopted instead of the burdensome method of wheel concentrations that had been in vogue for the preceding ten years. This paper received a very thorough discussion, from which it was evident that bridge engineers and railroad engineers, as a whole, would be glad to settle upon a few standard loadings, and to adopt some simple equivalent method of computing stresses. Most of those who desired the abandonment of the "Concentrated Wheel-load Method," advocated the adoption of the "Equivalent Uniform-Load Method," but a few favored either the "Single" or the "Double Concentration Method," with a constant car-load.

This paper, with the discussions, was published in the February and March 1892 number of the *Transactions* of the American Society of Civil Engineers, and was reviewed very generally by the technical press, attention being paid principally to the subject of equivalent loads. These reviews started a series of letters by the author and others, that were printed at first in the *Railroad Gazette*, and later also in the *Engineering Record*, in which letters the subject of equivalents was thoroughly and exhaustively treated. These proved that the "Equivalent Uniform-Load Method" gives results which are accurate enough for all practical purposes, and that neither the "Single Concentrated-Load Method" nor the "Double Concentrated-Load Method" gives results coinciding at all closely with those found by the theoretically exact method of "Wheel Concentrations."

In November 1892 the author sent a circular letter to all the chief engineers of railroads in the United States and Canada who were members (in any grade) of the American Society of Civil Engineers, and to every other member of that society connected with or specially interested in the designing, building, or operating of railroad bridges. This letter solicited a ballot on certain "Disputed Points in Railway Bridge Designing," foremost among which were those of standard live loads and a simple equivalent method for computation. The number of responses received was as great as could have been expected; and the result was that about eighty-two per cent of those who voted favored and eighteen per cent opposed the adoption of "a Standard System of Live Loads for Railway Bridges" similar to that proposed by the author. Eighty-two per cent also of those who voted were in favor of abandoning the "Concentrated Wheel-Load Method," and eighteen per cent were in favor of retaining it. Of the former, seventy-eight per cent favored the "Equivalent Uniform-Load Method," and twenty-two per cent were in favor of either the "Single" or the "Double Concentration Method." A number of gentlemen who responded made valuable suggestions in respect to the standard system of live loads propounded, and by the aid of these the author prepared a proposed "Compromise Standard System of Live Loads for Railway Bridges," and submitted the same, as before, for final ballot in May 1893.

The number of replies received showed that great interest was taken in the question; and the result of the ballot was ninety per cent in favor and ten per cent opposed to the proposed standard.

Next the pamphlet was published and distributed quite generally among those engineers interested in the subject of bridges, a copy being sent not only to every one who had replied to the ballots, but also to every railroad chief engineer in the United States, Canada, and Mexico whose address was given in Poor's Manual. To these chief engineers there was also sent another circular letter with a ballot that read as follows:

I Agree / Do Not Agree to use the "Compromise Standard System of Live Loads for Railway Bridges" when calling for bids on railroad-bridge work, or when having plans prepared for railroad bridges.

I Agree / Do Not Agree to specify that the "Equivalent Uniform-Load Method" is to be used in computing stresses in the bridges that are to be designed for my road.

Signature of Voter.

....

Chief Engineer of the

....

Over one hundred chief engineers thus addressed voted in favor of both propositions, and very few were opposed.

The pamphlet has now been in use more than four years, and has been in such demand that the first edition (a large one) has been exhausted. All those who have used its methods indorse heartily both the loads specified and the Equivalent Uniform-Load Method.

METHOD OF UTILIZING THE EQUIVALENT LOADS.

In calling for bids on bridge-work to be accompanied with designs for the structures, a railroad engineer can nominate any bridge specifications whatsoever, standard or otherwise, and at the same time specify that the live loads are to be taken from the "Compromise Standard System," and that the "Equivalent Loads" thereof are to be employed.

In this "System" will be found from "Class Z" to "Class T," inclusive, a close approximation to any live load that an engineer is likely to want to use; and if, for a certain car-load, some engineer should prefer a heavier or lighter engine-loading, he can obtain practically what he wishes by specifying that one class is to be used for floor systems and primary-truss members, and another class for main-truss members. The author does not advise this, however, except in the case of double-track bridges, where it would be advantageous to use a certain class for floor systems and primary-truss members, and a lighter class for the trusses, because the chances of there being two, full, maximum train-loads on the span at the same time are generally very small. It might be well to carry this idea even further by specifying, for instance, "Class V" for stringers, "Class W" for floor-beams and primary-truss members, and "Class X" for main-truss members of double-track bridges. Such a method would be in accordance with the theory of probabilities; but it would not apply to single-track bridges, for which the locomotive and car loads of the "Compromise Standard System" have been properly adjusted.

The "Equivalent Uniform-Load Method" reduces to a minimum the labor of making computations of stresses in bridges. The correctness of this statement will be rendered evident by the ensuing explanations of the use of the method. As for its exactness, if any-one has any doubt whatsoever about its closeness of approximation to the theoretically correct method of wheel-concentrations, let him read the author's letter in the *Railroad Gazette* of July 28, 1893. An inspection of Table I of that communication shows that no reasonable man can object to the "Equivalent Uniform-Load Method" because of its want of exactness.

In designing a bridge, one commences naturally with the stringers, then passes to the floor-beams; and afterwards to the trusses; so let us follow this order.

STRINGERS.

From Plate III find the equivalent live load per lineal foot for a span equal to the panel length, add to same the assumed weight per foot of two stringers and the floor they support, and divide the sum by two, calling the result w; then find the total bending moment at mid-span by substituting in the well-known formula,

$$M = \frac{1}{8}wl^2,$$

where l is the panel length in feet, and M is the required moment in foot pounds.

Should the total end shear be required, it can be found for each stringer by adding together the end shear given on Pl. II and the total weight of one stringer with the floor that it carries, and dividing the sum by two.

FLOOR-BEAMS.

In proportioning a floor-beam, the important thing to ascertain is the total concentration at the point where two stringers meet. The live-load concentration is to be found by multiplying together the panel length and the equivalent uniform load per lineal foot given on Pl. III *for a span equal to twice the panel length*, and dividing the product by two. It is unnecessary to describe here how the dead-load concentration at each stringer support is to be found. Nor is it necessary to do more than merely mention that the live-load concentration obtained for the floor-beam is the same as that required in finding stresses in primary-truss members.

TRUSSES.

These can be divided into two kinds, viz., those with equal panels and parallel chords, and those in which the panel lengths are unequal, or the chords are not parallel, or both. In the first case, the stresses can be determined most expeditiously by substitution in tabulated formulæ, and in the second case by the graphical method.

Case I.

From Plate IV find the equivalent uniform live load per lineal foot for the given span length and multiply same by the panel length, calling the product L. For single-track bridges this must be divided by two. All the live-load stresses in main-truss members of single-intersection bridges can be found by substituting this value of L in Table XVII.

Just here it is proper to remark that the "Equivalent Uniform-Load Method" is not applicable to trusses of multiple intersection; but the most approved modern practice in bridge-engineering does not countenance the building of trusses or girders having more than a single system of cancellation. The "Equivalent Uniform-Load Method" does, however, apply to trusses with divided panels, such as the Petit truss; but as this style of truss nowadays involves almost invariably a polygonal top chord, its treatment herein will come under

Case II.

Where trusses have unequal panels or chords not parallel, the first step to take is the finding of all the dead-load stresses by the graphical method, starting from one end of the span and working towards the middle, where the last stress is checked by the method of moments, and the correctness of the entire graphical work is thereby proved.

The next step is to find from Pl. IV., as in Case I, the equivalent live-load per lineal foot for the span, and therefrom the value of the panel-truss live-load L. Next set a slide-rule for the ratio of dead load per lineal foot and the equivalent live load per lineal foot for the span, and, by referring to the dead-load stresses already found, read off from the rule all of the live-load stresses in chords and inclined end posts.

Next assume that there is an upward reaction at one end of the span equal to 1,000 pounds, 10,000 pounds, or 100,000 pounds (according to the size of the bridge), caused by a load placed at the first panel point from the other end of the span, then find graphically the stress in each web-member from end to end of span, caused by this assumed upward reaction.

Then calculate the value of the live-load reaction for the maximum stress in each web-member by means of the slide-rule and the following formula and table, in which n is the number of panels in the span, n' is the number of the panel point at the head of the train, counting from the loaded end of the span, and C is the coefficient of $\frac{L}{n}$.

Live-load reaction for the head of train at $n' = C \times \frac{L}{n}$.

n'	C	n'	C	n'	C	n'	C
1	1	7	28	13	91	19	190
2	3	8	36	14	105	20	210
3	6	9	45	15	120	21	231
4	10	10	55	16	136	22	253
5	15	11	66	17	153	23	276
6	21	12	78	18	171	24	300

Then, still using the slide-rule, find the greatest live-load stress in each web-member by the following equation:

$$\text{Stress required} = \text{Stress from Assumed Reaction} \times \frac{\text{Actual Reaction}}{\text{Assumed Reaction}}.$$

Where the panels are divided as in the Petit truss, and where inclined subposts are employed, the *tensile* stress in the *upper* half of each main diagonal thus found will have to be corrected by subtracting therefrom a stress equal to $\frac{L}{2}$ sec. A, where A is the inclination of the diagonal to the vertical. But when inclined subties are used instead of inclined subposts, the correction just referred to will apply only to the *compressive* stresses in the *lower* halves of the main diagonals. The reason for making this correction, as will be at once evident to any one who is accustomed to finding stresses in Petit trusses, is that the method above outlined ignores the subdivision of the panels when ascertaining by graphics the stresses caused by the assumed upward reaction.

In comparing the equivalent loads for spans of one hundred feet, given on Pl. III, with those given on Pl. IV, an apparent discrepancy will be noticed. This is due to the fact that Pl. III is for plate-girder spans, for which the equivalent loads were obtained from the bending moment at mid-span; while Pl. IV is for truss-spans, for which the equivalent loads are the average of those at all of the panel points.

CHAPTER XX.

TIMBER TRESTLES.

TIMBER trestles can be divided into two general classes, viz.: *First*, Pile-trestles, or those in which each bent is formed of several piles, a cap, and transverse sway-bracing; and,

Second, Framed Trestles, or those in which each bent is composed of squared timbers framed together and braced.

Owing to the excessive length of piles required for greater heights, pile-trestles should rarely, if ever, exceed thirty feet in height; while framed trestles, if properly designed for rigidity as well as for strength, may be carried up to much greater heights, the economic limit being probably about one hundred feet.

PILE-TRESTLES.

The bents of a pile-trestle should contain at least four piles each. Where the trestle does not exceed ten feet in height, the piles may be driven vertically, and no sway-bracing need be used, provided that the piles have a good penetration in reliable material. For greater heights of trestle than ten feet, the two outer piles of each bent should be given a batter of from two to three inches to the vertical foot. Each bent should also be braced with one or two sets of sway-bracing, each composed of two $3'' \times 10''$ yellow-pine diagonals, thoroughly bolted to the piles, wherever they cross them, by $\frac{3}{4}''$ bolts. Wherever the piles are of irregular sizes, they should be trimmed off so as to make the diagonal bracing fit properly.

The piles for such bents should be so spaced laterally as to give great transverse rigidity to the structure, and at the same time afford ample support for the caps. A good spacing is as follows: Distance centre to centre of outer piles, 11′ 0″; distance centre to centre of two inner piles, 4′ 6″.

The caps should be at least 12″ × 14″ × 14′, placed on edge and attached to the piles by means of $\frac{7}{8}$″ drift-bolts.

For ordinary pile-trestles in fairly firm soil no longitudinal sway-bracing will be required for heights below ten feet; but for heights between ten and twenty-two feet, single-deck, longitudinal sway-bracing should be used in every fifth panel, so as to prevent the structure from moving longitudinally as a whole because of thrust of trains. For heights greater than twenty-two feet, each alternate panel should be braced longitudinally by double-deck bracing, so as to hold the piles at mid-height, and thus strengthen them as columns; the transverse sway-bracing for these cases should also be double-deck for the same reason.

For ordinary pile-trestles up to twenty-two feet in height the panels should be a trifle less than fourteen feet in length, while for greater heights either the same length may be used or alternate panels may be made from twenty-four to twenty-eight feet long by trussing the stringers, according to which of the two methods is the more economical.

The stringers under each rail should be built of three runs of timber, generally sixteen inches deep, the sizes being determined from the loading and by using an intensity of two thousand pounds for the extreme fibre, when impact is included. The stringer timbers are to be separated from each other at the panel points by means of timber packing-blocks, which are to serve also as splice-timbers. These timber blocks should be at least three inches thick and six feet in length, and should have at least four bolts through them. They are to be separated from the stringers by small cast-iron fillers three quarters of an inch thick, so as to prevent the timbers from coming in direct contact with each other. The splice-timbers must be made wide enough to project an inch or two below the bottoms of stringers, and must be notched over the caps so as to hold the stringers firmly in place. The distance from centre to centre of middle stringers should be five feet. Intermediate cast-iron separators with bolts should be used between adjacent stringer-timbers, at distances not to exceed five feet centres.

The length of the stringer-timbers for ordinary trestles should be twenty-eight feet, so as to extend over two panels, and thus stiffen the floor system materially.

The ties should be 8″ × 8″ × 10′. They should be dapped over the stringers at least one-half inch and spaced thirteen inches from centre to centre.

Inside and outside guard-rails should be used for all trestles, and at each end of every trestle some satisfactory kind of rerailing device should be used. The outer guard-rail should be made of a 6″ × 8″ timber laid flat and dapped one inch on the ties. The inner faces of the outer guard-rails should be spaced not less than twelve inches from the gauge-planes of rails. The inner guard-rail should be 6″ × 8″, laid flat and dapped one inch on the ties. The outer faces of the inner guard-timbers should be placed five or six inches inside the gauge-planes of rails. Both inner and outer guard-rails should be bolted to alternate ties by three-quarter-inch bolts, which must pass through the stringers also. The heads of these bolts should be countersunk into the tops of the guard-rails by means of cup-shaped washers.

FRAMED TRESTLES.

For trestles of greater height than thirty feet, and for less heights under certain conditions, it will be necessary to use framed bents. The foundations for these may be provided by driving piles and cutting them off above the ground, by using timber sills, or by building small masonry piers.

In all such trestles it will be necessary to brace the structure thoroughly, both transversely and longitudinally.

All framing of bents should be done in such a manner as to tie all parts firmly together.

For very high trestles it will be economical to increase the lengths of alternate panels to twenty-five or even thirty feet, and truss the stringers.

The longitudinal bracing should consist of diagonals of timber of suitable dimensions, in alternate panels, with horizontal struts made continuous at bracing panel points throughout all panels.

In addition to the transverse and longitudinal bracing previously described, all trestles on sharp curves should be provided with a lateral system composed of timber diagonals spiked to caps and to bottoms of stringers.

What has been said in regard to flooring for pile-trestles applies also to framed trestles.

The compression-members, when impact is included in the stresses, are to be proportioned by the formulæ

$$p = 1700 - 0.4\left(\frac{l}{b}\right)^2$$

for long-leaf yellow-pine and hard woods, and

$$p = 1000 - 0.2\left(\frac{l}{b}\right)^2.$$

for white pine, short-leaf yellow pine, and soft woods, in which formulæ l and b are respectively, in the same unit, the unsupported length and the least transverse dimension of the strut.

SPECIFICATIONS FOR TIMBER TRESTLES.

CLEARING AWAY RUBBISH AND PREPARING GROUND FOR STARTING WORK.

Before beginning work on any trestle, all rubbish, logs, trees, and brush must be cleared away, and all combustible material must be burned or removed for the entire width of the right-of-way.

DIMENSIONS

All posts, braces, stringers, ties, guard-rails, sills, and all timber generally, shall be of the exact dimensions given and figured in the drawings. No variations from these will be allowed, except upon the written consent of the Engineer or his duly authorized representative.

DRAWINGS.

The drawings will be made to the scales indicated, but in all cases the figures are to be followed in preference to the scale, where there is any discrepancy between the two. The drawings are to be followed exactly, excepting in cases of errors or omissions, which must be referred to the Engineer for correction or for additional information.

TIMBER.

All timber shall be of good quality, and of such kinds as the Engineer may direct. It must be free from wind-shakes, wanes, black, loose, or unsound knots, sap, worm-holes, and all descriptions of decay, or any other defect which would impair its strength or durability. It must be sawed true and out of wind, and to exact dimensions. Under no circumstances will any timber cut from dead logs be allowed to be placed in any portion of the structure; but all timber must be cut from living trees.

PILES.

The piles are to be cut from good, live trees of such varieties of timber as may be selected by the Engineer. They must be straight, sound, and perfectly free from wind-shakes, wanes, large, loose, black, or decayed knots, cracks, worm-holes, and all descriptions of decay; and they must be stripped of all bark.

If square piles are to be used, they must be hewed square and not sawed, but must be as free as practicable from axe-marks. Square piles must be at least twelve (12) inches across the face, and must show not more than two (2) inches of sap across the corners.

The sizes of round piles will depend upon their length, but in no case shall they be less than nine (9) inches in diameter at tips. They shall be so nearly straight that a right line, taken in any radial direction and running parallel to a right line joining the centres of ends of pile, shall show that the pile is at no point over one third of its diameter at such point out of a straight line. All piles must show an even and gradual taper from end to end ; and the tip ends are to be pointed in an approved and workmanlike manner. Wherever the piles are liable to encounter logs, bowlders, or any other material which is liable to split or injure them, the ends are to be protected by cast or wrought iron shoes.

Whenever in driving it becomes apparent that the hammer is splitting or injuring the head of a pile to any material extent, the top is to be banded by a heavy wrought-iron ring while the pile is being driven.

All piles must be cut off at tops to an exact line so that the caps will bear evenly on all the piles of the group.

All piles injured in driving, or that are driven out of place, shall either be cut off or withdrawn, as the Engineer may elect, and others shall be driven in their stead.

Whenever the heads of the piles are of greater diameter than the width of the caps, they are to be adzed off at the tops at an angle of about forty-five (45) degrees, so as to be flush with the sides of the caps.

All piles must be accurately spaced according to plans, and those beneath the track-stringers must be driven vertically.

All battered piles must be driven to the angle shown on the drawings. Where piles of different diameters are used in the same bent, the large piles must be adzed off where the diagonal braces cross them, so that the diagonals will not be bent out of line.

FRAMING.

All framing must be done to a close fit, and in a thorough and workmanlike manner. No blocking or shimming of any kind will be allowed in making joints, nor will any open joints be accepted anywhere on the work.

All joints, ends of posts, ends of piles, etc., and all surfaces of timber which are to be placed in direct contact with other timber or with masonry, must be thoroughly painted with hot, creosote oil, and then covered with a good coat of hot asphal tum, or such other material or materials as may be selected by the Engineer.

All holes of any kind, which are bored in any of the timbers, are to be thoroughly saturated with hot asphaltum, and all bolts and faces of washers, which are to be placed in direct contact with the timber, are to be warmed and dipped in a vat of the same material.

The holes for all bolts of three quarters ($\frac{3}{4}$) of an inch or more in diameter are to be bored one eighth ($\frac{1}{8}$) inch less in diameter than that of the bolts which are to be used in them. For smaller bolts, the holes are to be bored one-sixteenth ($\frac{1}{16}$) inch less than the diameter of the bolts.

All caps are to be thoroughly drift-bolted to tops of piles. All bracing timbers are to be bolted to piles, caps, or other timbers wherever they cross them. The ends of all stringers shall be firmly attached to caps by means of drift-bolts, timber cleats, or some other method which, in the opinion of the Engineer, is equally good.

For structures on curves, the superelevation of outer rails is to be provided for by bevelling the ties, not to exceed three (3) inches in five feet, or by dapping inner stringers on caps not to exceed two (2) inches, or, where the required superelevation is too great to be provided for by either of the two methods named or by a combined use of them, by cutting off the tops of the piles on an inclined plane. The last method is not to be resorted to unless it be absolutely necessary, and then extreme care must be taken to cut the piles off so that their tops will lie in a true plane. In no instance is this to be done when framed bents are used, as the inclination can then be given in cutting the tops of the bent posts to receive the caps.

METAL-WORK

All bolts, nuts, and dowel-pins shall be made of soft steel or wrought iron of the same quality as that specified for adjustable members of bridges in Chapter XVIII. Preference will be given to screw-bolts of soft steel with cold-pressed threads.

All bolts must be practically perfect in every respect, and, wherever necessary, they must be provided with nuts and threads of the standard size required for their diameter. The thickness of a nut shall not be less than the diameter of the rod for which it is intended, and the side of a square nut must not be less than twice the diameter of the bolt. The heads of all bolts shall be of the same size as the nuts required for the screw ends. All screw-bolts, drift-bolts, and dowel-pins shall be made truly straight before being driven, and all nuts must be screwed up tight against the washers. All nuts and heads of bolts must have heavy O.G. washers between them and the timber. All washers are to be made of cast iron of good quality, and must be sufficiently large and thick to provide properly for distributing the pressure due to the greatest allowable tension in the bolts over the area of the washers. They must be finished in a neat and workmanlike manner, and must be free from air-holes, cracks, cinders, and other defects. All spacers are to be made of cast iron, unless otherwise specified, and must be of the same quality and finish as specified for the washers.

ERECTION, DEFECTIVE WORK, DIRECTIONS TO CONTRACTOR, CLOSING THOROUGHFARES, RESPONSIBILITY FOR ACCIDENTS, DAMAGES, ALTERATION OF PLANS, STRICTNESS OF INSPECTION, SPIRIT OF THE SPECIFICATIONS, ENGINEER, AND TENDERS.

See Chapter XVIII.

CHAPTER XXI.

INSPECTION OF MATERIALS AND WORKMANSHIP.

UNLESS all the materials used in a structure and all workmanship during the various stages of manufacture at the shops and of construction in the field be subjected to competent and honest inspection, much of the benefit obtained by scientific design and thorough specifications will be lost.

For many years most of the inspection of structural metal-work was a sad farce; and, in consequence, the general public placed but little confidence in inspection, with the result that a large portion of the bridge-work of the country was left entirely to the tender mercies of the manufacturers, who naturally worked for their own interest and not for that of the purchasers. Latterly, however, owing to the efforts of a few first-class inspecting bureaus, the status of inspection has been somewhat improved, although it is far from being to-day what it ought to be. In making this last statement the author speaks advisedly, in that he has suffered considerably, even of late years, from bad inspection in such matters as the insertion of a rust-joint in a turntable between the bottom of drum and top of upper-track segments, where no such filling was allowed in either plans or specifications; badly matching holes in field connections; pinholes too small for pins; important members omitted in shipping; eye-bars made longer than called for by the drawings; great recesses in webs and fillers at ends of girders; and shop-paint applied over half an inch of frozen mud. Such things, to say the least, are extremely annoying, and often cause great expense during erection.

Primarily, the blame for such errors must fall on the inspectors, for such egregious blunders should never escape their observation. But they are by no means entirely to blame for the fact that the inspection of structural steel in general is not what it ought to be; because back of them are the railroad managers and promoters of large enterprises, who do not recognize the necessity for first-class inspection, and who are often not willing to pay one half of what such inspection is worth. Here again, though, the inspectors are to blame, for the reason that in the keenness of their competition for work they have cut prices to such an extent as to make it impossible to do proper inspection without losing money. When pinned down to facts they have to confess this. The coolness of some of the "small fry" inspectors is often amusing The author was once hauled over the coals by one of this class who had put in a low bid for some inspection, and whose tender had been rejected because of the low figure, the work having been awarded to one of the regular inspecting bureaus at about fifty per cent more than the unsuccessful bidder asked. After expressing his mind pretty freely, he fired this parting shot: "Well, I never intended to do thorough inspection for you, anyhow."

The inspection business has been utterly demoralized in times past by just such action as that contemplated by this inspector; for it was the general custom, and is yet to a certain extent with some inspectors, to take contracts for inspection at whatever figures the purchasers are willing to pay, then handle the work so as not to lose money on the contract, regardless, of course, of the interests of their employers.

Strange tales concerning inspection come to the ears of engineers—such, for instance, as passing car-load after car-load of metal-work that was not seen by the inspector until after loading for shipment; but such tales need verification, which, of course, it is nobody's business to give them. There is no doubt, though, of some of them being authentic. In one case in the author's experience the inspector left his work for ten days *in charge of one of the bridge-company's shipping-clerks*, without notifying either the author or his direct employers, the inspection bureau, of his contemplated absence. Such actions as this make one entertain doubts sometimes as to whether inspection really pays.

It is possible that the general demoralization of metal inspection by insufficient prices and keen competition has lowered the quality thereof to such an extent that even the highest possible prices would not make it, for some time to come, what it ought to be; because not only are the assistant inspectors lacking in proper training and thoroughness, but the manufacturers have become accustomed to a certain class of inspection, and would deem it a hardship to be subjected to much more rigid requirements. Eventually, however, the resulting improvement in manufacture of metal-work would be an advantage to the manufacturers as well as to the purchasers.

A decided improvement in inspection can be brought about only by concerted action on the part of the principal inspecting bureaus and inspectors of the country, backed, of course, by the aid of all engineers who are directly interested in the designing and building of structural metal work. If these inspecting bureaus and inspectors of established reputation were to form an association for the purpose of determining what inspection should consist of, and what minimum rates should be charged therefor by all members of the association, and if admission to the association were based upon both experience and good faith, it would be practicable to make very quickly the improvements requisite for bringing inspection up to an almost ideal standard of excellence. For a while a good deal of work would go to the inspectors outside of the association; but ere long the general public would become educated to the fact that good inspection of metal-work is a necessity, and that it can only be obtained by paying living prices to those who do the work.

Engineers, in order to aid in the good work of the association, should refuse to include the price of inspection in their fees for engineering work, and should make it a rule to employ for doing their inspection only members of the association.

Certain engineers of high standing have spoken slightingly of this proposition to form an association of inspectors, terming it a "trust." Strictly speaking, it certainly would partake of the nature of a trust, but it would be a good and worthy one, whose main object would be to effect a much-needed reform. On the same basis the American Institute of Architects is a trust, for the reason that it establishes a minimum fee of five per cent for the making of plans and specifications and for the services of an inspector on all building work; and surely such an organization should not be condemned on this account. On the contrary, the architects have set the engineers a good example in forming this association; and, until engineers follow their lead in this particular and establish minimum fees for professional work, the engineering profession will fail to attain its highest degree of efficiency, and will therefore not be properly recognized as a profession by the general public.

Returning to the question of the inspection of structural steel, the author herewith presents, as his idea of what good inspection should consist, his standard instructions to the inspecting bureau which he employs and to its inspectors at mills and shops.

First. Study carefully the engineer's drawings as soon as they are finished, and make out a list of special points and features that will require extra care in the shops to secure good workmanship and proper fitting, then make out a typewritten report of these and submit it without delay to the Engineer.

Second. Study carefully, as soon as they are finished and approved, all shop drawings, so as to become thoroughly familiar with the entire construction.

Third. Make sure that metal of uniform character and of the strength, elasticity, and ductility specified is furnished by the rolling-mills, following the metal from one process to another from start to finish, and making sure that the test-pieces broken represent correctly the metal they are supposed to represent.

Fourth. Check the chemical analyses of the metal occasionally, so as to see that they are properly made, taking care that the Contractor is informed as to what piece the samples are taken from, so that he can make a check test, if he so desire.

Fifth. See that all the various tests indicated in the specifications are made faithfully, the number of same depending upon the relative uniformity of the metal furnished.

Sixth. Make sure that all the punching is done with such care that the assembled parts will come together so as to make the rivet-holes match so accurately that when the reaming is finished there shall be no irregular holes.

Seventh. Make sure that all pieces are cut to exact length and proper bevel, that all web-stiffening angles bear perfectly at top and bottom against flange angles, and that there are no loose rivets.

Eighth. Wherever rivets with flattened heads or countersunk rivets are called for, make sure that they are properly chipped or otherwise brought to correct dimensions; also see that the ends of all members are limited to the lengths beyond the last rivet or pin hole shown on the drawings. Give particular attention to the ends of all posts and chord-members to see that the "over-all" and the clear dimensions between jaws correspond faithfully to those indicated on the drawings.

Ninth. Take some effective means of ensuring that the entire work shall go together properly and without difficulty during erection, and so that when completed it shall conform in every particular with the Engineer's design, even if, to accomplish the same, it be necessary in special cases to assemble the entire work at the shops.

Tenth. Watch carefully the punching and handling of the metal in the shops, so as to see that no cracks develop therein, and that the metal withstands properly the manipulation, showing as perfect homogeneity as is found in the best structural steel.

Eleventh. Condemn, as soon as it is discovered, any material unfit in the slightest degree for use in the structure, no matter how many times it may have already been inspected and passed.

Twelfth. See that all metal-work is properly cleaned by the most approved methods and apparatus before the first coat of paint is applied, and that the latter is allowed to dry thoroughly before the metal-work is loaded on the cars for shipment.

Thirteenth. See that all shop painting is thoroughly done, and that proper paint, mixed so as to comply with the specifications, is invariably used; and make an occasional chemical analysis of the paint, taking care that the Contractor is notified of the contemplated test after the samples are taken, in order that he may make a check analysis, if he so desire. Take special care to prevent any pieces of metal from being riveted together, unless the contiguous faces be first thoroughly painted.

Fourteenth. Insist upon the discharge of any employee of the Manufacturing Company who wilfully violates or continues to violate the specifications and the instructions given by the Engineer or his inspectors.

Fifteenth. While endeavoring in every possible way to obtain good work, avoid as much as possible doing anything to annoy or harass the Contractor; but, on the contrary, take special pains to aid him in every legitimate manner to finish his work quickly and inexpensively.

Sixteenth. Formulate and prepare for each large piece of work the best practicable method of recording progress and reporting thereon, and divide up the total work into groups or sections, so that the notes may be easy for reference. This should be done by the inspecting bureau, and should not be left to the shop inspector.

Seventeenth. Send into the office of the Engineer regular weekly reports concerning the progress of the work, any special reports that from time to time appear to be required, the tabulated results of all tests of materials, and copies of all shipping bills.

Eighteenth. Make sure that all shipping weights are correct by seeing the metal weighed, and keep account of the weight of all metal sent out on the work, as the Contractor will be paid by the pound. It will be necessary for the inspecting bureau to check all of these weights against the shop drawings to show how they agree or disagree. A detailed statement of both sets of weights must be sent to the Engineer upon the completion of the contract, or, at his request, upon the completion of any definite portion thereof.

Nineteenth. The inspecting bureau shall, under no circumstances whatsoever, intrust responsible work of any kind to insufficiently trained assistants. When new inspectors are to be broken in, they must receive their training in such a way as not to jeopardize in the slightest degree the quality of the material or workmanship.

Twentieth. Finally, and in short, do all you can to make the structure in every sense of the word a credit to all concerned in its designing and construction.

The author has had made for him lately by Mr. R. T. Lewis, one of his inspectors, a rather interesting series of tests to determine the average accuracy of punched rivet-holes. These tests were made after the metal was assembled for reaming by inserting rods of various diameters in the assembled holes. From the results of these tests the author has prepared the following clause for the specifications given in Chapter XVIII.

"All punched work shall be so accurately done that, after the various component pieces are assembled and before the reaming is commenced, forty (40) per cent of the holes can be entered easily by a rod of one sixteenth ($\frac{1}{16}$) of an inch less diameter than that of the punched holes; eighty (80) per cent by a rod of a diameter one eighth ($\frac{1}{8}$) of an inch less than same; and one hundred (100) per cent by a rod of a diameter one quarter ($\frac{1}{4}$) of an inch less than same. Any shop-work not coming up to this requirement will be subject to rejection by the inspector."

It will be noticed that this specification does not reject absolutely all work that does not come up to its exact requirements, the inspector being allowed some latitude in distinguishing between simple and complicated shop-work, important and unimportant connections, and the assembling of few and of numerous component pieces.

If the Association of Inspectors herein suggested were established, it could do good work for the engineering profession by laying out a series of tests of full-sized members and details of bridges and other structural metal-work, to be made from time to time as a portion of the inspection for large contracts. This would need the assistance of the consulting engineers, who, in preparing their specifications, should include, as a part of the work of the manufacturers, the making, under the supervision of the inspectors, of certain tests of full-size parts, it being understood at the outset that the results of such tests shall be of direct value to the accomplishment of the work covered by the specifications. The author has for the past five years been endeavoring in this way to obtain some much-needed information concerning the strength of both main members and details of bridges and elevated railroads; but his attempts to have the tests made have not always proved successful.

As for the proper price to pay for first-class inspection, the author would state that some three years ago he submitted to several of the principal inspecting bureaus a draft of instructions to inspectors at mills and shops, similar to those incorporated in this chapter, with a request that they tender upon inspecting for him, according to said instructions, a large order of structural steel; and that the bids received varied from one dollar to one dollar and twenty-five cents per ton of two thousand pounds. Subsequent experience has proved to the author that such inspection as he then called for is worth fully one dollar per ton for large orders and a trifle more for smaller ones; although it is very seldom that such a price is paid in this country for inspection.

In respect to inspection of materials and workmanship in the field, the following instructions, which the author has prepared for his field forces of engineers and inspectors, will be found to cover the subject pretty thoroughly.

(A) METAL-WORK.

First. Examine with the greatest care all of the metal-work as fast as it is delivered, so as to make sure that it has not been injured during transportation, and keep an eye on it thereafter to see that it is not injured during erection. See also that there are no missing parts.

Second. See that the metal-work goes together properly and expeditiously, and report to the Engineer all necessity for chipping or filing on account of bad shop-work.

Third. Watch carefully the riveting to see that no burnt rivets are used, that all field-rivets are driven in accordance with the specifications, and that no loose rivets are left in the work.

Fourth. See that all vacant spaces in the metal-work are completely filled with paint-skins or other water-proof material before the painting is begun.

Fifth. In elevated-railroad work see that during the erection of the metal-work the lengths of the girders are sufficiently correct to prevent all possibility of using up the spaces provided for expansion, assuming the greatest temperature of the metal to be one hundred and twenty-five degrees. See also that the expansion and contraction of the structure cannot injure the stairways.

Sixth. In drawbridges, see that the masonry of the pivot-pier is levelled off with the greatest accuracy, and that the lower track-segments are set to exact position and level, thus making a perfectly conical surface for the rollers. See also that the latter are adjusted so as to bear evenly at top and bottom against both upper and lower track-segments.

Seventh. See that the ends of draw-spans are properly adjusted by means of the shimming-plates on the rest piers and those in the bottom chords near the pivot-pier. Make sure that in every particular the draw is reversible end for end; and see that all shafting is properly aligned so that there will be no binding in any of the bearings.

Eighth. See that, before the operating machinery is tested, all sliding or rolling surfaces are thoroughly lubricated, and that the turntable is cleared of all obstructions, such as nails, etc., on the lower track-segments. Then make a test of the machinery and compute therefrom the horse-power required to operate the draw.

Ninth. See that all anchor-bolts are set in exact position and to correct level, and that they are properly grouted in.

Tenth. In placing the bearings for arches, take the greatest care that the centres are set to exact position and level, and that the bearings for the metal-work on the masonry are perfect.

Eleventh. Whenever there are any adjustable rods used in a structure, see that they are properly tightened before the work is left, taking care that they are not screwed up more tightly than is really necessary.

(B) RAILS.

First. Examine all rails as soon as received, so as to see that there are no poor ones which have escaped the rail-inspector's eye, or which have been loaded for shipment after being rejected. Inspect also all other track-metal, such as angle-bars, bolts, and braces, so as to see that they are of the correct type and are delivered in good shape.

Second. See that all rails are laid to exact line and level, and that they bear properly everywhere.

Third. In draw-spans, make sure that the track-rails at the ends will not interfere with the operation of the draw.

(C) PAINTING.

First. See that, after proper cleansing and retouching with paint, the metal-work receives its first field-coat of paint as soon as practicable after erection, and that the next coat is applied as soon as practicable after the first field-coat is thoroughly dried, but in no case before.

Second. Make sure that all paints used are of the proper color, quality, and consistency, and that no adulterants or thinners are used; also, that all paint is properly applied.

Third. Look carefully to the painting of all close spaces between metal, and see that it is done effectively with a piece of cloth, according to the specifications.

Fourth. See that all portions of the metal-work, which are to rest on the masonry or which are to be embedded in the concrete, receive their two field-coats of paint in due time, so as to dry thoroughly before the said metal-work is erected.

(D) EXCAVATION.

First. Watch carefully all excavation so as to make sure that it is done in strict accordance with the specifications and with the City Ordinances, if there be any. See that, in doing the excavation and in building the structure, the Contractor does not obstruct public traffic.

Second. In foundation-work in cities, see that all pipes and sewers are moved properly and coupled or spliced effectively after being uncoupled or cut.

Third. Whenever there is any doubt about the proper resistance of any foundation, test it by loading it by means of a properly designed and built apparatus. Always ram thoroughly any foundation where the resistance to load would be increased by such ramming. See that the material from the sides of the pits is prevented from falling in.

Fourth. See that all surplus material is removed expeditiously from City streets, and that, whenever any piece of construction is completed, all falsework, rubbish, etc., are removed from the site and are deposited in an unobjectionable place.

(E) FOUNDATIONS.

First. See that the bed-rock is always properly prepared to receive the caisson or masonry, as the case may be, letting the caisson into the rock so as to provide an even bearing around the cutting edge, and levelling or stepping off or filling up with concrete to receive the latter.

Second. In elevated-railroad work, see that wherever columns are located in the street their feet are properly encased in concrete, and that cast-iron fenders are correctly set around the columns and filled with concrete and grouting, then sealed effectively against the ingress of water. See also that, after the columns are up and encased, the pavement is relaid in a substantial manner, to the satisfaction of the City authorities.

Third. When large steel cylinders are used, see that they are kept well braced with timbers on the inside during sinking, so as to avoid all possibility of collapse.

Fourth. See that proper guides are provided for all caissons and cylinders, so that they can be kept in exact horizontal position during the entire sinking.

Fifth. See that the tops of all piers are properly finished off to receive the superstructure, taking care that all bearings are made perfectly smooth and to exact level.

(F) CAISSONS.

First. In building timber caissons, see that the plans are followed exactly, and that the full quantum of timber bolts is used; also, that short timbers are not put in where long ones are called for. See that all timbers are properly framed.

Second. In sinking caissons, see that they are never allowed to deviate materially from correct position, and that all errors of position are corrected as soon as possible after they are discovered.

Third. In filling working-chambers of caissons, see that the concrete is packed tightly against the roof, and that no voids whatsoever are left therein.

(G) CEMENT AND CONCRETE.

First. Test all the cement, according to the special instructions therefor, so long before it is needed for use that the Contractor shall not be delayed by such testing.

Second. See that all cement is housed so as to be protected effectively from the weather, and that no dampened or otherwise injured cement is allowed to be used on the work.

Third. Inspect as soon as delivered, and if possible before it is dumped on the ground, all sand and broken stone, so as to make sure that they comply in every particular with the specifications; and insist always upon all of these materials that are rejected being removed immediately from the vicinity of the bridge site.

Fourth. See that strong and proper forms for concrete are used in the construction of all pedestals, and that all visible portions of the latter are finished off smooth, the top surface being brought to exact elevation and made perfectly level.

Fifth. See that all concrete is mixed according to the specifications, that it is put in place immediately after mixing, and that it is thoroughly rammed.

Sixth. See that no injury is done to the concrete in removing the timber forms, or, if any be done, that it be properly repaired ; also, that the timber be left in whenever its removal would tend to injure the work.

Seventh. When concrete is placed under water, see that either a trémie or proper collapsing-bucket be used, and that the water be not permitted to injure the concrete. See also that all such concrete is mixed extra rich.

(H) PILING AND TRESTLEWORK.

First. See that all piles conform in size, quality, and straightness with the requirements of the specifications, even if they have been already passed by the timber inspector before shipment, and reject any that are unfit for use.

Second. See that all piles are driven straight and in proper position, and that the tops are not unduly injured in driving, having the said tops banded, whenever necessary to prevent splitting.

Third. See that all piles are cut off at the exact elevation required, and that the caps are properly drift-bolted thereto. On curves see that the superelevation is obtained properly, and not by shimming up on the caps.

Fourth. See that all sway-bracing is bolted effectively to the piles and caps.

(I) TIMBER, FLOORING, AND HAND-RAILS.

First. Inspect all timber as soon as delivered, marking plainly all rejected pieces ; and see that all such pieces are removed from the vicinity without delay, in order to prevent their being put into the structure without the knowledge of the resident engineer. It is, of course, permissible to use the good portions of rejected timbers; but in doing so great care should be exercised to prevent the workmen from putting any poor material into the work. The fact that all the timber received had been previously accepted by the timber inspector is no reason for using unsatisfactory material; moreover, sometimes it happens that timbers which the inspector has never even seen are marked with his stamp and shipped.

Second. See that the floor system is properly laid and attached to the metal-work, that each rail bears effectively upon every tie which it crosses, and that the rails are laid straight, evenly, and to exact grade.

Third. See that the hand-railing is brought to proper alignment, and is held there in a permanent manner.

Fourth. See that all joists in highway bridges are properly dapped on floor-beams so as to bring all of their upper surfaces to exact elevation or elevations; also, that all intermediate joists lap past each other far enough to reach entirely across the top flanges of the floor-beams. See that the outer lines of joists abut and run continuously, and that they are effectively spliced on the inside.

Fifth. See that all joists in which the depth exceeds four times the thickness are bridged at distances not to exceed eight feet, and that when the hand-railing depends for its rigidity upon that of the outer joists the latter are well bridged and otherwise stiffened where the posts are attached.

Sixth. See that alternate bolts attaching guard-rails to floor pass through both the flooring and the outer joists, and that all holes through the latter are bored in the central plane of the joist.

(J) MASONRY.

First Inspect all stone as soon as received, so as to see that it has not been injured in transit, and that it is satisfactory in every particular, even if it has already been passed by the stone inspector.

Second. See that all stones are thoroughly cleaned and wet before being laid.

Third. See that all mortar is mixed in the proper proportion, and that it is used on the work before any set has occurred.

Fourth. See that all joints are thoroughly flushed with mortar, and that no voids are left anywhere in the masonry.

Fifth. See that all coping-stones are set so that the top of the pier will lie in a truly horizontal plane, and that they are kept in place by proper clamps and dowels as per plans.

Sixth. See that the exposed joints are all cleansed and pointed in a thorough and workmanlike manner, and in accordance with the specifications.

(K) GENERAL INSTRUCTIONS.

First. See that all proper precautions against accidents to the public and to the workmen be taken during erection, and that no glaringly careless man be allowed on the work.

Second. If there be more than one Contractor on the work, see that no friction arises between contractors, and that their combined work is finished in good shape.

Third. While doing everything in your power to obtain good work, avoid as much as possible worrying or harassing the Contractor, and use every legitimate endeavor to aid him to complete his work expeditiously and inexpensively.

Fourth. Finally, and in short, study the specifications carefully, and do all that you can to insure the structure's being in every respect a credit to all concerned in its designing and construction.

In respect to the testing of cement on construction work, the following instructions, which the author has prepared for his resident engineers, will give the reader all necessary information, it being understood that no brands of cement are ever used except those which either the author or his assistants have previously tested thoroughly by long-time tests, and which have proved to be perfectly satisfactory :

First. In testing cement in the field, remember that it is not a series of laboratory tests which you are to make, but that your object is simply to see that you are receiving and using cement of an average quality of the standard brand or brands adopted, and that it comes up to the general requirements of the specifications.

Second. Look out for irregularities in the quality of the cement, so as to avoid using any that is either too old or too fresh, or which has been injured by dampness.

Third. Test first for fineness, second for soundness, third for activity, and fourth for rise in temperature, rejecting all cement which is unfit for use because of non-compliance with the specifications in these particulars.

Fourth. It will seldom, if ever, be necessary to resort to the boiling test, which is essentially a laboratory test; although it may prove useful in an emergency to determine conclusively whether certain cement is fit for use or not.

Fifth. Test all cements for the tensile strength of neat briquettes, making one day and seven-day tests. Never pass cement on shorter time-tests than seven days, as the one-day test is by no means conclusive.

Sixth. Make, more for your own satisfaction than for any other reason, a few sand-briquette tests for seven and twenty-eight days, so as to know the value of the mortar which you are using. It would not do to rely on sand-briquette tests for the acceptance or rejection of cement, as this would delay the work too much.

Seventh. You will often have to use your judgment about passing or rejecting cement that is needed for immediate use and which fails in some comparatively unimportant point to quite fill the requirements of the specifications. Rather than delay the Contractor materially, pass such cement, provided that in your opinion its use will in no way injure the quality of the work; but, on the other hand, if the rejection of the said cement will not delay the Contractor seriously, insist on its complying with the specifications in every particular. Be careful not to let the Contractor run in any poor cement or force it upon you because of any assumed or real necessity for haste in completing the construction.

In respect to inspection of stone for masonry, the author offers, as his idea of what stone-inspection should be, the following instructions to stone-inspectors, it being understood that they apply only to stone from quarries that have been previously investigated and found satisfactory:

First. Reject all stone containing any dry seams. These seams are often very hard to detect; but usually by a careful inspection of the surface of the stone they may be found. Sometimes a mere line is all the evidence of the existence of such seams, while in other cases they show more plainly.

Second. Reject all stone containing seams called "crow-foot," which are either open, or which are liable to dissolve out after exposure to the weather.

Third. See that no stone is quarried at a time when it is liable to freeze before the quarry-sap is out of it. Stone should be quarried at least a month before it is allowed to freeze.

Fourth. See that no powder or other explosive is used in quarrying the stone, excepting to remove ledges of useless stone, and even then make sure that no stone to be used is injured by the explosives.

Fifth. If the stone be of such a character that the quarry-bed cannot be told at a glance, the Inspector must mark each stone in such a manner that it will be sure to be laid in the wall on the said quarry-bed.

Sixth. Reject all stone which is taken from any portion of the quarry that is affected injuriously at any time by frost.

Seventh. See that all stone is handled carefully after being taken from the quarry, so that no cracks are developed or other injury done thereto by rough usage.

Eighth. See that all stones are cut to the exact dimensions called for by the plans, and that they comply in every particular with the specifications.

In respect to inspection of timber, both in the woods and at the sawmills, the author's instructions to his timber-inspectors, as follows, will be found useful:

First. Study well and compare with the mill people all order-bills, looking carefully to the various lengths, widths, thicknesses, bevels, numbers of pieces, etc., so as to make sure that your order-bills check properly against those furnished to the mill people and against the partial order-bills furnished by the latter to their various employees, so as to avoid all possibility of errors. If any be found, correct them yourself, if possible; but, if not, refer them to the Engineer for correction.

Second. Each timber-inspector is to be provided with a special stamping-hammer of his own, that has a characteristic mark which will identify all timber passed by him. He is to keep this hammer at all times in his own possession, so that it can be put to no illegitimate use by interested parties; and under no circumstances is he to lend it to another inspector.

Third. Each timber-inspector must study carefully the specifications furnished him, and must be governed thereby; nevertheless, there will be occasions when he must trust to his own judgment as to what timber is fit and what is unfit for the required purpose, for general specifications cannot be made broad enough to cover all cases that may arise in filling a timber bill. Where a number of inspectors are employed on the same piece of work, it will be necessary at the outset for the Chief Inspector to interpret the specifications and supplementary instructions for all of the assistant inspectors, so that the latter shall not differ at all in their requirements.

Fourth. In inspecting timber be careful to distinguish properly between the various varieties that are fit and those that are unfit for use. If not otherwise stated in the specifications, you are to accept and reject as follows:

OAKS.

Accept white, cow, chincapin, post, burr or overcup, and live oaks. Reject red, Spanish or water, black, black-jack, and pin or yellow butt oaks.

PINES.

Accept white, Norway, long-leaf Southern yellow, short-leaf yellow (for certain purposes only), and Cuban pines; also Oregon fir. Reject Southern red, loblolly, and Rocky Mountain yellow pines.

CYPRESS.

Accept red, black, and yellow cypress. Reject white cypress.

Fifth. Secure timber of as uniform a character as possible, avoiding any that shows large heart-checks or growth-checks, and rejecting any which has such defects within one inch of face or edge of timber. Avoid all coarse-growth, open-grained timber, if other timber be procurable.

Sixth. Reject any sticks that show signs of worm-holes, decay, scorching by forest fires, ring-heart, ring-shakes, rotten or black knots, dark or discolored spots, or any other defect that would impair the strength or durability of the timber.

Seventh. Examine carefully by probing with a wire all hollow or bird-eye knots, and, should the hollow be over one inch deep, reject the timber.

Eighth. Check lengths of cutting gauges every day, as they are liable occasionally to be knocked out of position. Check widths and thicknesses at each change of the machine.

Ninth. In inspecting piles, look carefully to their straightness, and see that they comply in this and in every other particular with the specifications.

Tenth. See that due care is used in handling and loading timber so as not to bruise it; and under no consideration allow it to be floated in the water after it is cut and dressed.

Eleventh. Keep a daily record of all timber accepted, so that the Engineer may be informed on short notice as to how much of any bill has been cut.

Twelfth. Notify the Engineer or other proper party of all shipments, and keep an accurate account of everything shipped, so that upon short notice a statement in respect to any uncompleted order can be made, giving the amount that has been shipped and the amount that remains to be forwarded.

Thirteenth. The Chief Inspector must make regular monthly reports to the Engineer or other proper party or parties concerning the progress of the work, quality of timber furnished, etc.; and must send in monthly statements of all moneys received and expended by him in connection with his work of inspection.

Fourteenth. Use every endeavor not to cause by your inspection any more handling of material than is necessary for doing your work thoroughly; and do nothing to give the mill people needless worry or expense.

In concluding this chapter, the author desires to emphasize his previous statement that, in order to obtain a truly first-class structure, it is necessary not only to design it properly and prepare thorough specifications for its building, but also to provide a corps of competent and honest inspectors, who will from start to finish examine carefully and test all materials that are to be used, and who will see that the entire manufacture and erection are done in strict compliance with the specifications.

CHAPTER XXII.

DESIGNING OF PIERS.

THE object of this chapter is not to provide the bridge-builder with either a complete specification for building piers of all kinds or full directions as to sinking them under all possible circumstances, but to indicate to the designer, first, how to determine the best kind of piers to use at any proposed crossing, and, second, how to proportion them. Text-books on substructure do not generally cover this ground, but deal mainly with masonry specifications and methods of sinking piers. The reader who desires to learn anything concerning piers which is not given in this chapter is referred to Baker's "Treatise on Masonry Construction" and Patton's "Practical Treatise on Foundations."

In determining the layout of spans for any important crossing, the first question to settle is what method of pier-sinking to adopt, for upon this will depend to a certain extent the span lengths.

The three principal methods in common use are as follows:

1. The Coffer-dam system.
2. The Pneumatic process.
3. The Open-dredging process.

The use of coffer-dams is, or should be, limited to crossings where the bed rock is not more than fifteen feet below the ordinary stage of water, and where there is no great, sudden rise anticipated. This method always figures low in the preliminary estimate, but is generally found to run much higher when the total cost of the finished structure is computed. The author nearly always discourages his contractors from attempting to use this method; and thus far his experience proves that, when they fail to adopt his advice about it, they are generally sorry therefor by the time the work is finished.

Coffer-dams are liable to give trouble in several ways: first by leakage, second by flooding, and third by collapsing. If a Contractor gets through a large piece of coffer-dam foundation work without accident or trouble of some kind he is in great luck.

For bare bed-rock, movable coffer-dams may be employed; but they are troublesome to construct, and are sometimes very difficult to remove because of a deposit of sand taking place while the piers are being built.

The pneumatic process for sinking piers is in most cases the best one to employ, the only objection to it being the excessive cost of installing the plant, even if one has a complete outfit at his disposal. Its great advantages are that it enables the contractor to overcome, in the cheapest and most expeditious manner possible, all obstacles that may be encountered in sinking; and that it ensures the obtaining of a satisfactory foundation for the piers. It can be used for depths as great as one hundred feet or even more, although there is considerable danger to the workmen when the depth exceeds eighty or ninety feet.

Most of the bridge-piers which the author has put in have been sunk by the pneumatic process; and he has no hesitation in recommending it as the most satisfactory, all-around method in probably nine cases out of ten which occur in a consulting-engineer's practice.

The open-dredging process is suitable for very deep foundations, or for putting down caissons that are to rest on the sand, or for bed-rock foundations where there is no liability of great scour. For large piers this process is much cheaper than the pneumatic on account of both the smaller cost of plant and the more rapid progress in sinking. In case, however, that obstacles be encountered, such as trees or large boulders, the expense for sinking is liable to run high, as these obstacles may have to be removed by a diver or divers, which always involves great expense. The author has put down three large piers by the open-dredging process, two to a depth of ninety feet and one to a depth of one hundred and twenty-two feet below extreme low water, and has encountered no trouble worth mentioning during the sinking. In the case of the greater depth, a mass of boulders was found overlying the bed-rock. This was penetrated as far as practicable by excavating the boulders and laying bare the bed-rock near the centre of the pier, then firing charges of dynamite at the bottom till the cylinder refused to sink any farther, after which it was filled up with concrete.

It is probable that, if one were to try to sink small piers to any great depth by the open-dredging process, difficulty would be experienced because of the lack of sufficient weight of pier as compared with the large amount of skin friction. The latter in sand is generally a little less than six hundred pounds per square foot of vertical surface. On the East Omaha bridge the author arranged to reduce this friction by means of small water-jets placed around the circumference of the cylinder about every six feet in height; but these were found to be unnecessary, so were not used. In case of striking clay or any other sticky substance, such an attachment might prove of great service.

The open-dredging process is liable to abuse by the builders of cheap highway bridges, who, in order to save a little in first cost, use it to sink cylinder piers of small diameter moderate distances to bed-rock, which may in these places be laid bare or nearly so by excessive scour. With this process it is generally not practicable to anchor the cylinders firmly to the bed rock, but with the pneumatic process it is.

There is still another style of foundation besides the three described, viz., that which involves the use of piles. These piles may either support a timber grillage, upon which to rest the pier proper, or may run up into the concrete body of the pier. This class of foundation is of a cheap order, but will often answer the purpose very well, provided there be no possibility of excessive scour. If the bearing capacity of the piles be small, it is best to spread them out and cover them with a thick timber grillage; but otherwise it will be found economical to run the piles up into the concrete. The author has lately designed the piers for three important Southern bridges in the latter manner. They were put down without much difficulty, the principal hindrance being from sunken logs, and were eminently economical in first cost as compared with piers of other possible designs.

Piers may be divided into the following classes in respect to the materials of which they are built:

1. Stone-masonry piers resting on—
   A. Bed-rock.
   B. Timber and concrete caissons.
   C. Steel and concrete caissons.
   D. Timber grillages supported on piles.
2. Brick-masonry piers resting on the same foundations as mentioned for stone-masonry piers.
3. Unprotected concrete piers resting on the same foundations as mentioned for stone-masonry piers.
4. Oblong steel shells filled with concrete and resting on the same foundations as mentioned for stone-masonry piers.
5. Cylinders filled with concrete and resting on—
   A. Bed-rock.
   B. Timber and concrete caissons.
   C. Timber grillages supported on piles.
   D. Piles.
6. Braced steel piers resting on—
   A. Bed-rock.
   B. Masonry, brick, or concrete piers.
   C. Cylinder piers.
7. Timber piers resting on—
   A. Mudsills.
   B. Piles.

Now in respect to which of these seven kinds of piers and which of their various supports it is best to adopt for any particular crossing, the engineer must use his judgment, which, however, may be aided by the following remarks that are based upon the author's experience:

Class No. 1.

Masonry piers should be used for important railroad bridges and for very large highway bridges, where first-class stone can be obtained at reasonable cost. If good stone is not obtainable at a fair price, it is better to use one of the other classes. The proper way to proportion a masonry pier is to determine the least size under coping to support either the pedestals themselves or the pedestal-blocks, as the case may be, leaving a small margin on the exterior and ample room between pedestals or pedestal-blocks to allow for variation in erection, then batter the pier all around not less than one-half inch to the foot, or as much more as investigation shows to be necessary. The coping should project all around about six inches, the amount depending upon the magnitude of the pier and the thickness of the coping course, which should be from eighteen to twenty-four inches.

The batter for the sides is to be determined in the following manner: Compute for both the loaded and the empty structure the greatest longitudinal components of the total wind-pressure that can come upon the pier from the two spans which rest thereon, upon the assumption that the friction at the roller ends of the spans is zero.

The direction of the wind which will give the greatest longitudinal thrust on the structure is forty-five degrees in respect to its longitudinal axis. As the cosine of this angle is approximately seven tenths, the longitudinal component of the wind-pressure per lineal foot of span will be seventy per cent of the assumed said wind-pressure.

Find also the greatest traction thrusts from braked trains on the assumptions that, first, the greatest live load is on the structure, and, second, that the least live load, or one thousand pounds per lineal foot, is on same. Now find the values of the following combinations:

1. Thrust from wind load on empty bridge.
2. Thrust from heaviest braked train.
3. Thrust from wind load with lightest live load on the spans.

4. Combined thrust from lightest possible braked train, and a wind-pressure on train and structure equal to one half of that specified.

Next determine by judgment the proper batter, and lay off the pier to scale; then divide it by horizontal planes from four to six feet apart, and compute the weights of all the portions of the masonry between these planes, making a proper reduction for weight of water for those parts which would be submerged by an average stage of river.

Next compute the wind-pressure on each vertical division of the pier, down to the assumed stage of water, in a direction parallel to the spans, using the same intensity and direction for the wind-pressure as were adopted in finding the longitudinal thrust from wind-pressure on the spans.

Next find graphically for all four cases the curves of pressure from the vertical and horizontal loads at top of pier, combined with the weights of the various divisions of the latter and the wind-pressures thereon, and see that none of the said curves at any plane of division pass outside of the middle third of the section at said plane. If any of them do, the batter will have to be increased, or, if all the curves fall much inside of the middle third points, it will have to be decreased; and in either case the graphical computations will have to be made again, and so on until a satisfactory batter is found.

The author is aware of the fact that this method of designing piers is not in general use, and it is quite possible that he is the sole engineer who adopts it; nevertheless he maintains that it is the only proper way to design masonry piers. The single concession which he would be willing to make on the score of economy would be to assume that a certain small portion of the thrust on a span is taken up at the roller end. But if the rollers are in good working order the amount of thrust that they will resist is very small indeed—so small, in fact, that the author prefers to neglect it entirely.

The ordinary method of proportioning piers is to make them as small as possible under coping and batter them all around, or at least on the sides, one-half inch to the foot. In some cases this will suffice, but in others it will not. One of the largest bridges in the United States has piers built with such insufficient batter that it is evident at a glance, to even an untrained eye, that something is wrong. By the way, one of these piers is cracked from top to bottom, owing to false economy in the design, but not because of its failure to figure properly for the curve of pressure.

An inherent sense of fitness in the mind of the designer will generally tell him, when he looks at a scale-drawing of the superstructure and piers of a bridge, whether the latter are properly proportioned. In the case of the Red Rock cantilever bridge over the Colorado River the piers were first laid out fourteen feet wide under coping, with a batter of half an inch to the foot, and the drawings were submitted to the author for his criticism. He immediately pronounced the piers to be proportioned incorrectly, simply because of their appearance. Their proportioning was then turned over to him, and he found by trial that a batter of one and a quarter inches to the foot was necessary. This batter gave a satisfactory appearance to the entire layout.

In nearly every case the length of the piers up and down stream, determined by the minimum size under coping and the proper side-batter for thrust, will provide sufficient strength and stability to resist both current and wind pressure. A thorough investigation of resistance to overturning of piers down-stream is given in Baker's "Treatise on Masonry Construction." In it he proves that any pier which is large enough under coping, and which has ordinary batter, will resist properly the overturning tendency of the worst possible combination of loads from wind, current, and floating ice. Nevertheless, in long-span, single-track bridges with very high piers, crossing swift streams that carry thick ice, and where the structure is exposed to high winds, it is advisable, as a matter of precaution, to test the piers for down-stream overturning according to Prof. Baker's method. Should the length of pier parallel to the stream be found insufficient, the neatest way to obtain the requisite stability is to put in a cocked-hat just above the elevation of extreme high water.

Where a masonry pier rests on bed-rock, the latter should be levelled or stepped off, and there should be placed a layer of rich concrete between the rock and the masonry.

If an ice-break with an inclined cutting edge be necessary for any pier that rests on a yielding foundation, a corresponding ice-break or similar offset should be placed at the down-stream end of the pier also, even if its appearance be as incongruous as would that of a cowcatcher at the rear of a railway train; for, unless the foundation be thus balanced about the centre of gravity of the vertical load, the portion directly under the superstructure will tend to settle more than that under the nose, and will thus cause a cracking of the masonry and a splitting-off of the front of the pier. Such a disastrous result of the violation of the principle of symmetry in designing is by no means unknown, even in important railroad bridges.

In respect to timber-and-concrete caissons for masonry piers, the following general remarks will prove useful to the designer:

There should be an offset of not less than two feet all around the base of the masonry, and preferably a little more at the ends, so that in case the caisson be located a little out of place the masonry can be shifted thereon so as to bring the pier into proper position. The number of courses of 12″ × 12″ timber in the roof of the caisson should never be less than four and seldom more than eight. Any less number than four would be liable not to give the roof the proper stiffness during the sinking, and any more than eight would tend to cause an undue settlement of the pier on account of the compression of the timber, which always takes place. The designing of the roof and sides of the working-chamber should be done with the greatest care, so as to prevent all possibility of collapse, and the cutting edge should be shod with steel plates to protect the timber when the caisson is passing through boulders or logs. The roof-timbers, if possible, should always be of full length, and the spacing of the bolts therein should not exceed four feet. The vertical timbers on the outside of the working-chamber should be carried well up into the roof, shouldered, and firmly bolted thereto. The crib above the working-chamber should be sheathed so as to reduce the friction during sinking. The drift-bolts should be seven-eighths-inch rounds driven into three-quarter-inch bored holes. The filling of the working-chamber with concrete should always be done with the greatest care, using extra rich concrete, so that there shall be no voids between the concrete and the roof. Portland-cement of the best quality should be used for filling the working-chamber and shafts; but it is legitimate to employ an extra-good quality of American natural cement for filling the crib in case that it be necessary to keep down the expense. However, Portland cement is always preferable.

In respect to caissons built of steel and concrete but little need be said, except that great care should be taken to design the working-chamber strong enough to resist properly the weight of the concrete above and the unequal pressures from boulders below. The metal below the roof of the working-chamber should not be less than one-half inch in thickness, and all parts near the cutting edge should have thicknesses varying from three quarters of an inch to an inch. All joints in the cutting edge should be full spliced, as should also those in the roof of the working-chamber.

Timber grillages resting on piles should have, preferably, not less than four courses of timber, although often but three and more rarely two are employed. As the grillage is generally wider than the masonry, it takes about four courses to distribute the weight uniformly (or nearly uniformly) over the piles. In case of an unusually wide grillage, more than four courses would be necessary, or else the masonry should be widened by means of footing-courses. Such grillages should be built with care, so as to have a level bottom; and all piles should be cut off to exact level, otherwise there will be unequal bearing between piles and grillage that might cause serious damage to the masonry.

Brick piers are not common in America, probably because, until lately, it has been difficult to obtain proper brick. In the author's opinion, piers built exclusively of hard-burned clinker brick and mortar of the very best quality of Portland cement, mixed in the proportion of one part cement to two parts sand, and having thin joints perfectly filled, are better than the average masonry pier, for the reason that the bricks will never disintegrate, while the average stone used for bridge-piers will. The author has not yet had occasion to build any brick piers; but he intends to give them a trial on the first opportunity.

Unprotected concrete piers are satisfactory for Southern rivers, where the effect of frost is not severe, and where there is no ice of any account carried by the stream. The author used this style of pier for the Arkansas River bridge of the Kansas City, Pittsburg, and Gulf Railroad near Redland, Ind. Ter. Several other Southern bridges have piers of this type, and thus far they have proved satisfactory. Their chief recommendation is their cheapness. In order to ensure their being properly built, nothing but the best qualities of cement and sand should be employed, and the mortar for the concrete should be mixed rich, especially near the exterior of the piers. Some engineers give the work a skin-coating of rich mortar; but the author prefers to use finely broken stone and extra-rich mortar for six or eight inches all around, and to not attempt to smooth down the exterior. Of course it is practicable to put on a skin-coat so that it will stay, and so that it will not have a streaky appearance; but to do this requires more care than the average workman is inclined to take.

Steel shells filled with concrete make very satisfactory piers, provided they be not used in salt or brackish water, which would rust them out in a short time. These piers are applicable where good stone for masonry is expensive, and where the piers must be protected from the abrasion of ice or from the excessive cold, which would tend to disintegrate even fairly good concrete. Such piers can be built in the usual form of masonry piers with rounded ends all the way up, or, when they pass much above high water, they may run off into two cylinders with bracing between. Butt-splices are preferable, and the splice-plates below the mud line should be placed on the inside so as to offer as little resistance as possible to sinking.

This style of pier is a favorite one of the author's, for the reason that it is both sightly and inexpensive. When taken to task for using it, as often happens, he replies, "Good concrete protected with steel is better than poor masonry."

In respect to the thickness of steel to use, the author's practice is to adopt half an inch below the ordinary stage of water and three eighths of an inch above, although for cheap bridges he occasionally shades these thicknesses one sixteenth of an inch.

For the coping of such piers stone may be employed; but it is preferable to put on a moulding of sheet metal, as this is more in keeping with the rest of the pier. This style of coping has been criticised on the plea that it is false, and that it has no direct function; nevertheless, the author considers it eminently proper to use it, and that its function is simply to beautify the construction by relieving the harsh outlines. Where stone coping is not used, the top of any kind of concrete pier may be finished off with either rich concrete of small broken stone or with granitöid, mixed in the proportion of one part of Portland cement, two parts fine granite screenings, and three parts of small crushed granite.

Cylinder piers filled with concrete are the most common kind of pier in America, and they are certainly the worst; nevertheless they have their place in good construction, when they are properly designed and built. Their abuse is due mainly to the builders of cheap highway bridges, who think that if the top of the cylinder is simply large enough to hold the pedestals, that is all which is necessary, no matter how high the piers may be, how great may be the scour, or what kind of foundation there is. If piles are employed as a foundation, they put in all that their small cylinders will hold, and never dream of its being necessary to figure how many tons each pile will have to sustain.

Cylinder piers are legitimate construction in places where, under the worst possible conditions in respect to scour, they will have a firm grip in solid material, say not less in depth than twenty per cent of the height of the entire pier.

Cylinder piers will not often stand the test of the curve of pressures herein described for masonry piers; but this is not necessary, because they can resist tension on one side in both the metal and the concrete, if the latter be of the correct quality; i.e., the cylinders can act as beams to resist the horizontal thrust of wind and trains in the same way as do the columns of elevated railroads. Nevertheless for railroad bridges the author would advise against the adoption of long cylinders for piers, on account of their inability to resist vibration effectively. In some cases it is economical to adopt a group of four comparatively small cylinders well braced on all four faces; but with this style of foundation it is generally customary to employ braced piers resting on the cylinders.

The diameter for a cylinder should depend not only upon the size required at the top, but also upon its height and the character of the foundation. It is sometimes governed also by the total vertical load to be carried, which should under no circumstances exceed the limit set in the specifications given in Chapter XIV. Portland cement only, and that of the very best quality, should be used for filling cylinder piers, and the filling should be done with the greatest care and thoroughness. Whenever the concrete has to be placed below water it should be done by using a trémie, and the composition of the concrete should be much richer than that for concrete laid in the dry.

Whenever a cylinder is sunk to bed-rock, it should be let into same far enough to prevent all possibility of slipping, and so as to give an even bearing all around the circumference. This is an easy matter when the pneumatic process is employed for sinking, but it is often difficult when open dredging is used. This precaution is as necessary in the case of wooden or steel caissons as it is for cylinder piers, and should never be neglected where there is a possibility of scour to or near bed-rock, or where the pneumatic process is employed.

In sinking large cylinders by the open dredging process so as to fill them afterwards with piles, it sometimes becomes necessary to put in temporary timber bracing bolted to the metal, in order to prevent the cylinders from collapsing or from getting out of shape. Most, if not all, of these timbers will have to be removed before the piles can be driven.

It is economical sometimes to increase the diameter of a cylinder between top and bottom, but in such cases the lower twenty feet should be made plumb so that the cylinder can be sunk with ease and accuracy. This detail was adopted for the Jefferson City bridge, the variation in diameter being obtained by telescoping some of the lengths and putting in filling-rings. This required a trifle more metal than truly conical piers would ; but the shop-work was much simpler.

The proper load for large piles inside of cylinders is about thirty tons each, although with very large piles and solid material to hold them it may be increased to forty tons. On the other hand, in case of bad foundations, it is sometimes necessary to reduce the load to ten tons per pile.

The proper distance for piles to project into cylinders is about fifteen feet, and should never be less than ten feet or more than twenty feet under any circumstances. With less than ten feet there will not be enough grip for the concrete, and with more than twenty feet it is difficult to place the concrete properly under water between the piles. Piles in cylinders should be driven as closely as possible, and precaution should be taken to prevent the lifting of piles already driven by the sinking of the last few piles. When large piles are employed, the designer can figure upon six square feet of pier section for each pile.

The bracing between the up-stream and the down-stream cylinders of a pier should invariably be of solid webs properly stiffened, extending from high water to near low water, in case there be any drift ; but for cylinder piers on shore an open bracing of struts and ties will suffice.

Concerning braced steel piers but little need be said, except that they should conform in their design with the specifications given in Chapters XIV and XVI. It is advisable, if practicable, to avoid battering more than two faces of a braced pier on account of the troublesome shop-work that would be involved with a four-face batter ; nevertheless it is often necessary to adopt the latter, especially for high piers. A possible objection to this type of pier is that for cantilever bridges it increases the deflection of the span because of the compression of the pier columns under load. An extreme instance of such compression is that in the Niagara Cantilever bridge.

Timber piers are merely a makeshift, so do not merit much consideration. They are employed sometimes to support steel bridges until money is available for building masonry piers. It is seldom that timber piers are built in large rivers where the current is rapid and the scour is great. The author was once forced by circumstances into building pile piers under these conditions; and although they are still standing, he would sleep better at certain seasons of the year, had they never been built. The piers referred to are the temporary piers of the East Omaha bridge over the Missouri River. They were constructed in the winter, mostly on the sand-bar, by driving groups of seventy-foot red-cypress piles fifty feet into the sand by means of a powerful water-jet, then sheathing the sides and nose with four-inch oak planks and bracing the piles on the inside. The nose of each pier is on an incline, faced with steel plates where the ice can reach it, and forming a cutting edge that is capped with a heavy railroad rail. Each pier is surrounded with a woven willow mattress, eighteen inches thick, of the most substantial character, sunk and kept in place with rock. These piers have received a much more severe test than was anticipated when they were designed, because the channel has shifted across the river, so that at times there are thirty-five feet of water where there was a dry sand-bar when the bridge was constructed. The mattresses have not been injured by the scour, but have simply been lowered, the edges going deeper than the portions near the piers. The only ill effect noticeable is the springing down-stream of the tops of two piers, in one case about six inches and in the other about eleven inches. In order to bring the tops of these piers partially back to place and prevent any further deflection, the author employed a detail which has proved to be very satisfactory. It consists in passing one end of a strong iron chain loosely around an up-stream pile and dropping the loop to the bottom, then attaching near the other end of the chain a steel rod with an adjustment. A number of these chains were used for each pier, the rods passing through heavy timbers at the rear of the pier near the top. By screwing up on these adjustments the tops of the two piers were moved back a little. Provision is made for future scour by leaving some spare chain beyond the point at which the rod takes hold, so that one chain at a time can be loosened, lowered, and retightened. These East Omaha bridge-piers will probably last a long time yet, although when they were put in no one anticipated that they would be needed for more than eight years.

In sinking piers the greatest care should always be taken to start them in exact position and to keep them there. The instant it becomes evident that a pier or cylinder is getting out of correct position, it should be moved back, even if it be necessary to stop the sinking entirely until the true position be recovered. Generally it is feasible to build a frame of piles and heavy timbers around each pier or cylinder, so as to guide it to exact position at all times, barring a slight springing of the piles, which, however, can generally be guarded against.

Some sixteen years ago the author had occasion to sink four eight-foot cylinders in the Des Moines River by open dredging to bed-rock, so as to form a single pier, the axes of the cylinders being located on the corners of a twenty-four-foot square. Unfortunately the author in making the design, owing to inexperience, had provided no allowance for variation of location. The foreman of construction informed him that it was absolutely impossible to sink those four cylinders so correctly that the struts would fit between their tops; consequently the author was compelled to undertake the superintendence of the work himself. He built a strong frame of piles and heavy timbers, all thoroughly braced, around the space to be occupied by each cylinder, cutting out the horizontal timbers to fit the curve of an eight-foot circle, and even gouging out places for the rivet-heads to pass. One of these horizontal guides was located close to the surface of the water, and the other some nine or ten feet higher. The cylinders were dropped into these guides and sunk to bed-rock. After all four cylinders were in place, and partly filled with concrete, the struts were inserted between their tops, and were found to furnish a driving fit. This result was, perhaps, due as much to good luck as to good management; but the experience taught the author a lesson which he has never forgotten, and which he desires to impress upon all young designers, viz., that in preparing any substructure design it is essential to provide liberally for all possible variations from correct position in all parts of the work.

In respect to the designing of pedestals for elevated railroads and the determination of the bearing capacities of soils, the reader is referred to the author's before-mentioned paper on Elevated Railroads published in the 1897 *Transactions* of the American Society of Civil Engineers.

CHAPTER XXIII.

TRIANGULATION.

THE necessity for extreme accuracy in the triangulation for piers of long bridges is not generally recognized; hence result errors in pier location that sometimes require the lengthening or shortening of the superstructure, or which involve the adoption of an unanticipated skew. There is no excuse whatsoever for any such errors in location, because the method of triangulation adopted should provide a check against not only blunders, but also even trifling variations from correctness of position, and because the Contractor should invariably, at the outset of his work, take such precautions as will prevent the occurrence of any variation in sinking in excess of that provided for in the Engineer's plans.

In the triangulations for bridges over large rivers, such as the Missouri, the author makes a practice of measuring each base-line five times and each angle thirty times; and no point is ever located without using a check from another base-line, thus providing an intersection of three lines, which theoretically should be a mathematical point, but which actually varies therefrom, generally about a quarter of an inch, and sometimes even as much as one half of an inch, in sights of about one thousand feet length.

The author has tried both iron rods and steel tapes for measuring base-lines, and has adopted the latter as the more accurate. The objection to using rods is that it is almost impossible to run a line a thousand feet long with three rods that must always be made to actually touch each other without sometimes disturbing slightly the position of two of the rods, when either lifting or putting down the third rod. With [illegible] the average length [illegible] This would [illegible] measurement show a greater [illegible] of an inch in the [illegible] and another measurement [illegible] This tension is comparatively [illegible] If the ground be very [illegible] greater variation may be allowed. It should [illegible] per thousand feet.

[illegible] a new one for each structure. [illegible] bridge shops in comparison with [illegible] As a matter of precaution, it is well to test it [illegible] tape that is to be set aside as a reserve [illegible] happen to the primary tape.

For very long and important bridges, especially cantilevers with long spans, it would be well to have the tape tested by the Bureau of Weights and Measures at Washington, D. C., or by some other testing bureau of recognized standing—such, for instance, as that of the Washington University at St. Louis, Mo. The charge for such testing is usually merely nominal.

As the coefficient of expansion is not the same for all tapes, it might be advisable for extremely accurate work to have the coefficient determined for the tape to be used; but in most cases of long span bridges this would be an unnecessary refinement.

A fifty foot tape is long enough, and is in many respects preferable to those of greater length. The author has no use for extremely long tapes to measure distances directly between pier centres either during sinking or after the piers are finished, because this method of measurement is by no means as accurate as that of intersecting three lines on each pier and using two [illegible] measured base lines. The only direct measurement that is of any real value, and which can be obtained [illegible] the falsework is up, is one made on the ice. In such a [illegible] not to let the tape touch the [illegible] perfect line into holes therein and cut off to exact level. There is no more difficult measurement to make correctly than one with a long steel tape between two distant points without intermediate supports; because, in the first place, the double measurement on shore and in correct position is a slow and tedious one to make, involving as it does the use of the level to obtain the sag, which must be exactly alike in both cases; and, in the second place, the conditions of wind and temperature are likely to vary to such an extent as to cause errors that are very difficult to correct.

All base-line measurements should be made in cloudy weather, or just after sunset, or even at night: and the temperature should be noted for each fifty feet measured, as all lengths must be reduced to those for an assumed standard shop temperature of seventy degrees Fahrenheit. Even slight variations of temperature will cause errors of importance in the length of an ordinary base-line, the change in length per degree of temperature and per unit of length being about 0.0000066. For a base-line of one thousand feet and a variation of one degree the change in length would be eight one-hundredths of an inch. This, it is true, is no great amount, but there is always a liability of there being a difference of as much as ten degrees between the average temperatures for measurements made on two different days, and as much as two or three degrees in a single measurement of a base-line.

In using a steel tape it is better to start from the one-foot mark rather than from the end, unless the ring be placed back of the zero-point.

The author's method of measuring a base-line on comparatively level ground is to run in a line of stakes of at least three inches by one inch section and from two feet upward in length, spaced at intervals of about ten feet and put in to exact line and level, with a large flat-headed tack driven to line on each stake, and the true base-line scratched with a knife along the top of each tack. The line is measured by stretching the tape with a uniform pull of six pounds over the line of stakes, keeping the one-foot mark or the zero-mark, as the case may be, over the centre that is cut on the hub at the end of the base-line, and scratching with a knife on the tack where the fifty-foot mark on the tape comes, then starting from this point to measure another forty-nine or fifty feet, and so on until the centre of the hub at the far end of the base-line is reached. The next time that the line is measured the first length should be thirty-nine or forty feet, so as to avoid using the same tacks; and each succeeding first length should be ten feet shorter. This not only involves the use of fresh tacks for each measurement, but also prevents any manipulation of the tape so as to make the partial measurements agree with those made previously.

In case that a perfectly level line cannot be obtained, the line should be divided into level stretches, and where each break occurs the length should be measured on the incline and corrected afterwards for the effect of the rise or fall so as to obtain the true horizontal distance.

For further directions as to measuring base-lines with a steel tape, the reader is referred to Johnson's Surveying.

The ends of base-lines, as well as all intermediate points from which triangulation operations may be conducted, should be marked by solid and secure hubs. In protected places these may consist of six-inch by six-inch timber, three feet or more in length, driven in the ground and cut off about an inch above the surface, the centre being marked with a tack, across which are cut two intersecting lines at right angles to each other.

If the ground be subjected to hard freezing, the timber should be increased in section to eight inches by eight inches, and the length should be such that it will penetrate the ground, if possible, about three feet below frost. The earth around the hub location should be excavated to the greatest depth of frost, then the timber should be driven in or sunk like a post and well tamped, after which a stout timber box with an open bottom and a strong cover should be placed around the hub, and the earth should be packed around the outside thereof. Finally, the box should be filled nearly to the top of the hub with sawdust or dry sand.

In case that the ground be very hard, or if the bed-rock be near the surface, it will be best to surround the hub with concrete, and protect it with a substantial cover of some kind to prevent displacement.

If driving or carting is to be carried on in the vicinity of the hub, the latter should be fenced in by four stout posts sunk into the ground on the corners of a square of seven or eight feet on a side, the posts projecting high enough above the ground to strike a wagon-box.

In locating all triangulation-hubs it is essential to place them so that the operations of construction will not obstruct the view of the transitman.

If there is a possibility that any of the hubs will be disturbed by the operations of construction or in any other manner, such hubs should be carefully "tied in" by reference points located some distance away. This should be done as soon as the base-line is measured.

There should be two base-lines, one on each side of the river and both on the same side of the bridge, or both should be on the same side of the river with one above and the other below the bridge. Usually it will be found satisfactory to locate all piers from one point on each base-line, and for that reason the ends of the base-line should be chosen so that, if possible, all the piers can be seen therefrom. If this be impracticable, or if some of the deflections would for any reason be too small, it will be necessary to put in and use intermediate hubs on the base-lines.

Base-lines, whenever it is practicable, should be run approximately at right angles to the longitudinal axis of the bridge; but this is by no means essential, and it is folly to try to make the intersection exactly at right angles, except in the following case, which represents an ideal system of triangulation that can rarely be utilized, on account of the existing conditions of shores, and obstructions both natural and artificial.

The said ideal system consists in running four base-lines, as shown in Fig. 8, all exactly at right angles to the centre line of the bridge, and laying off thereon distances equal to those from the base-line to pier centres, so that all lines of sight will intersect the centre line at angles of exactly forty-five degrees.

The advantage of this system lies in the fact that all the piers are located by direct sight without having to measure the angle, the only angles requiring measurement being the four right angles between the base-lines and the centre line of bridge, and the four other angles required for determining and checking the distance between base-lines along the bridge tangent.

The lengths of base-lines for ordinary systems of triangulation will generally be regulated by local conditions. They

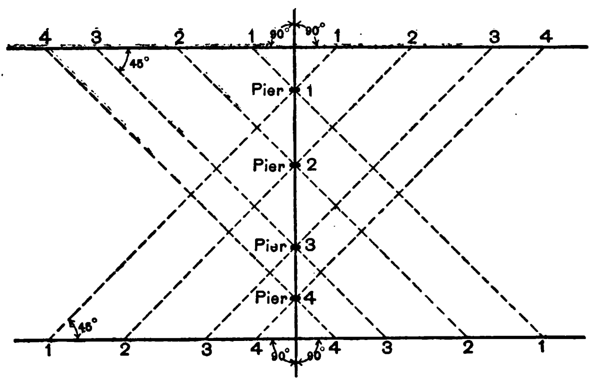


FIG. 8.

should usually be about as long as the total length of bridge, or, when there is a base-line on each side of the river, as long as the perpendicular distance between opposite base-lines; but, if necessary, they may be made as short as seven tenths of same. Too short base-lines will give too sharp intersections, and therefore sometimes too great variations from correctness; nevertheless, sharp intersections can be employed at times by taking extra pains with the work and by employing an extra intersection as a check, in case that any discrepancy occurs.

After the base-lines are measured and the hubs are put in, the next step to take is to measure the six principal angles of the triangulation. These should be measured with the greatest accuracy continuously around the limb of the transit, making from ten to thirty readings of each angle, according to the degree of refinement required. The instrument should be graduated for accurate work as fine as twenty seconds, or preferably ten seconds. A heavy transit with a good, solid tripod will usually give better results than those obtained by using a lighter instrument. The sun should never be permitted to shine on the instrument when the angles are being observed, as it is impossible to make accurate measurements under such a condition.

In keeping notes of triangulation-work a record should be made of the date, the temperature, the condition of the weather, the direction and approximate velocity of the wind, and the names of the transitman and picketman.

If long sights are to be taken, the picketman should be provided with a pair of field-glasses to enable him to see the transitman's signals; otherwise much time and labor may be spent to no purpose. Long sights should never be taken towards the sun when it can be avoided.

The error of all three angles in each of the two main triangles should not exceed two seconds in important work. Of course it is not necessary to go to any such refinement in short-span bridges; but in very long ones the error might well be reduced as low as one second. If the error in a triangle be found too large, it may be possible to avoid measuring all three angles again by looking over the notes and ascertaining from the weather conditions which angle is most likely to be at fault, then measuring this angle anew. If the second average angle reduces the total error in the triangle to within a proper limit, all right; but if not, the other two angles will also have to be measured a second time.

On the same principle, if, in a group of measurements of one angle, one or two readings be found to differ greatly from the others, they may be thrown out when obtaining the average.

It sometimes happens that both intersections of the bridge tangent with the base-lines cannot be seen from one end of one of the latter. In this case it will be necessary to put in a hub on the bridge tangent far enough ahead of the hidden point to clear the obstruction, triangulate to it, and measure the exact distance from it to the hub on the base-line. This expedient was necessary in the triangulation for the author's Jefferson City highway bridge.

A check on the accuracy of the triangulation work is obtained by comparing the two computed lengths of the bridge tangent between the intersections thereof with the base lines, or between one such intersection and a fixed point on the tangent on the other side of the river. The disagreement in these two measurements should be within the limit of one half of an inch to the one thousand feet. To show how accurately such work can be done, the author would state that for the Jefferson City bridge he gave his resident engineer instructions to allow no variation from correctness exceeding three eighths of an inch in either the main triangulation itself or in the intersections for pier centres. His instructions were followed so faithfully that no error exceeding three sixteenths of an inch was allowed to pass in any part of the work. The whole field-force once lost an entire half-day in rectifying an error of one half of an inch in the intersections for a pier centre. This is an excellent record for accuracy, considering that the distance between base-lines on the bridge tangent was a little over fifteen hundred feet. The author is generally not so rigid in his requirements for exactness as he was in this case, the reason for such strict instructions being the fact that this was the resident engineer's first experience in important triangulation.

The triangulation for the author's Sioux City bridge, made by Lee Treadwell, Mem. Am. Soc. C. E., with a bridge tangent about twenty-two hundred feet long between base-lines, was probably just as accurate as that for the Jefferson City bridge, because the errors in distances between pier-centres measured on top of the falsework were actually inappreciable.

After the main triangulation for a bridge is finished, the next step is to compute the angles to the various points on the piers that will be needed during the sinking. For a single cylinder pier it will suffice to triangulate to the centre only, and for a pier composed of two cylinders a triangulation to the centre of each cylinder will be enough; but for a rectangular pier it will be necessary to locate not only the centre, but also another point near the periphery, in order to prevent the pier from being rotated about its vertical axis in going down. After the calculations are completed a triangulation-sheet should be prepared, on which should be shown all of the triangulation with the various distances on all lines and the exact angles for all deflections.

Foresights should next be located for the bridge tangent and for all pier points, so that the transitman shall never be under the necessity of turning off an angle when locating a pier. The position for any foresight is generally determined by convenience, but it should be chosen so as to avoid any probability of disturbance. Each foresight, which consists of a substantial wooden target, 'is located by turning off the proper angle from the base-line, and is then fixed immovably in position, after which a series of from ten to thirty readings of the angle is made, the corresponding centre lines being marked on the target. The average of all of these centre lines is then determined, and is assumed to be the true centre, which is marked conspicuously on the target. Each target is to be marked also with its characteristic letter or number, so that its individuality may be recognized by the transitma. from the most distant point of observation. The angles for determining the correct centre of any target should be laid off continuously on the limb of the transit. All foresights should be inspected occasionally so as to see that they have not been disturbed, although any disturbance will be discovered, the first time that the foresight is used, by the three lines failing to intersect in a point.

When piers are to be built in open coffer-dams, the work of locating them is comparatively simple; for when they are once located little or no movement takes place afterwards. But when piers are to be sunk by the pneumatic process or by open dredging, great care must be taken at every step, because the pier is always either moving or liable to move at any moment. In sinking piers by either of the two last-mentioned processes, the resident engineer should keep such notes that from them he can report daily as to the exact horizontal position of the cutting edge of the caisson, the position of the top of the pier, the elevation of the cutting edge, the inclination of the axis of the pier to the vertical, and the amount, if any, that the pier has been revolved around its vertical axis. The Contractor can conduct his operations with much more certainty of landing the pier in its true position, if he be kept informed as to its relative position every day.

If temporary staging be used around the pier, from which to conduct the operations of construction, keeping track of the various motions of the pier will be a comparatively easy task, for the approximate alignment can be obtained from temporary points located on the staging, which points, however, need occasional checking to see that the staging has not shifted slightly.

If there be no staging, all locations will have to be made by triangulation, and, as before stated, two points on each pier will be needed in order to detect rotation. When the caisson has reached a considerable depth, however, the liability to rotate is greatly lessened.

After all that may be said, the work of keeping the pier in correct position will be dependent on local conditions and many varying requirements.

In respect to the levels, care should always be taken to preserve such measurements as will enable the leveller to keep a record of the vertical distance from the cutting edge to the top of the crib at each of the four corners. This will be necessary in order to determine how much the said cutting edge is out of level.

In giving the final elevations for the copings of the piers, it will sometimes be found necessary to take very long foresights, owing to the impracticability of setting up the level near the piers. In such cases a backsight should be taken to a bench-mark about the same distance from the instrument as the pier is therefrom, and in the opposite direction, so as to offset a possible slight lack of adjustment in the level, and to compensate for the curvature of the earth.

CHAPTER XXIV.

OFFICE-PRACTICE.

As there has been almost nothing yet written concerning the way in which work is handled in a Consulting Engineer's office, the author has concluded to close this little treatise with a chapter on "Office Practice"; and as no two engineers pursue exactly the same methods, and as the author is naturally more familiar with his own than with those of others, he will deal herein solely with the established practice of his own office, which practice is the outcome of over ten years of special effort to secure the best possible results both expeditiously and economically.

LAYING OUT WORK.

This chapter being confined entirely to office-work, it will be assumed at the outset that all such field data as profiles, maps, plats of borings, etc., have been secured.

In bridge-work it is necessary to determine the following:

First. The Purpose for which the Structure is to be used.—This being settled, there ensues the fixing of the live load, the clearance between trusses, and the clear height above base of rail or surface of roadway.

Second. The Clear Height between Standard High Water and the Lowest Part of Structure.—If the stream be a navigable one, the minimum clearance will be regulated by the requirements of the War Department. In other cases the clear height will depend on the required elevation of grade of railroad or roadway, provided that the lowest part of the superstructure will never offer any obstruction to floating drift or ice during the highest floods. The minimum clearance should preferably be ten feet, and never less than five.

Where a low bridge is required over a navigable stream, some one of the various kinds of movable bridges described in Chapters IX and X must be used; but for all ordinary cases the rotating draw is the most suitable type.

Third. Best Span Lengths to adopt.—In many cases there will be no choice as to span lengths, which are liable to be determined by such conditions as the requirements of the War Department, obstruction of stream by piers, danger from washout during erection, etc.; but, where the designer has any choice in the matter, he should be governed by the principles of economy laid down in Chapter III, taking care, however, that he does not violate any of the principles of æsthetics given in Chapter IV, unless he be forced to do so by circumstances that are absolutely beyond his control. As stated in Chapter III, the greatest possible economy will exist when the cost of each pier is equal to one half of the cost of the trusses and lateral systems of the two spans which it helps to support. The determination of these economic conditions is, of course, a matter of cut and try; but after a few trials the economic span length can be approximated very closely. In making such calculations the trial weights of trusses and laterals can be found with sufficient accuracy by taking a span of known weight and computing therefrom the weights for the spans of the trial lengths by the following methods:

A. The weight per foot of the lateral system is directly proportional to the span length, provided that the superstructure is not changed in width, which is generally the case. Should the width be changed, the new weight will have to be modified accordingly, under the assumption that the weight varies about half as rapidly as does the width.

B. To find the truss weight W' per lineal foot of span of length l' from the corresponding known weight W of span l, the following approximate but quite accurate empirical formula may be used:

$$W' = \frac{W}{2}\left[\frac{l'}{l} + \left(\frac{l'}{l}\right)^2\right].$$

This will give approximately the weight per foot of trusses for any span length, provided the live load per lineal foot remain unchanged.

C. To find for any span length the truss weight T' per lineal foot for a total load p' per lineal foot from the corresponding known weight T for a load p, the following approximate empirical formula may be used :

$$T' = \frac{T}{5}\left(1 + 4\frac{p'}{p}\right).$$

This is quite accurate for all ordinary spans, but for very long ones it gives too great a variation between T' and T.

After finding the value of T', the value used for p' should be checked; and if there be any serious disagreement between the value assumed and that found, the substitution in the formula should be made anew, and so on until a satisfactory agreement between the said values of p' be obtained.

Fourth. General Layout of Structure.—The general layout should consist of a profile, a plan, and enough cross-sections to illustrate properly the entire substructure, superstructure, and approaches, all being made to exact scale. For long crossings, a scale of one fortieth of an inch to the foot is the most satisfactory, but for short crossings the scale should be made larger.

The proportioning of the skeletons of the trusses should be done in accordance with the suggestions given in several of the preceding chapters, and the dimensions of the piers should be determined by the principles established in Chapter XXII.

Each general layout should give the following information

Elevations of bed-rock, low water, standard high water, extreme high water, lowest part of structure, grade-lines and tops of piers, lengths of all spans between centres of end-pins or centres of bearings, distances between centres of piers, all leading dimensions of piers, heights of trusses, and lengths and kinds of approaches.

As soon as the general layout is completed and finally adopted, the computations of stresses and sizes of members of spans may be begun.

For elevated railroads it is necessary to determine the following:

First. The number of tracks on the various portions of the line, and the clearances over streets and alleys.

Second. The live load per track to be carried by the structure.

Third. The location of the line, whether in the streets or on private property.

Fourth. The style or styles of girder construction. In some locations the City Ordinances may require open-webbed girders, as these shut out less light than do solid-plate girders, while in other locations the plate girders would be permissible.

Fifth. The location of columns, whether in the street or on the curbs, also, for location on private property, the number of columns per bent.

Sixth. The economic span length. As indicated in Chapter III, the greatest economy will exist when the cost of the longitudinal girders is equal to the cost of the cross-girders, columns, and pedestals. Where the columns are located in the street or on the curbs, due consideration must be given to the probable cost of removing underground obstructions, such as water-pipes, gas-mains, etc.

With these points all settled, the calculations for proportioning all parts of the structure may be proceeded with.

Where the structure is on a curve, it is best to make the bents radial whenever practicable. The exact location of each column should be figured from certain known lines, and all ordinates for same should be indicated on the layout. Much careful study should be given to the work of establishing each feature of the layout; for, if mistakes be made therein, they are liable to cause great delay and expense later on.

Roof-trusses and steel buildings will not be treated in this book, as it deals mainly with bridges, viaducts, and elevated railroads. The office work connected with the designing of roofs and steel buildings will, however, not differ essentially from that pertaining to the designing of the other structures.

CALCULATIONS.

After the leading features of any proposed structure have been determined, and after the general layout thereof is completed, the next step to take is the making of the calculations necessary to determine the stresses in all the parts and the proper sizes for same.

For convenience in making to correct scale pen-sketches of the various portions of the design, the author uses a cross-section paper divided into one-quarter-inch squares, the sheets being ten and a half inches wide by sixteen inches long, which size experience has shown to be the most satisfactory. At the head of each page are written the date, title of structure, and name of computer.

At the beginning of each set of calculations the following general data for spans are given:

First. Length of span.

Second. Number of panels.

Third. The various truss depths.

Fourth. Perpendicular distance between central planes of trusses.

Fifth. Live load or loads to be used.

Sixth. Wind loads for both upper and lower lateral systems.

Seventh. Spacing of stringers.

The dead load from the track and ties in railroad bridges or from the timber floor or pavement in highway bridges is first determined, using the unit weights of materials given in Chapter XIV; then the stringers or longitudinal girders are figured and proportioned, after which their weights and that of their bracing are computed.

Next the floor-beams or cross-girders are proportioned, and their weights are figured. From all these weights the weight per lineal foot of the metal in the floor system is next found.

As the lateral system can nearly always be designed before the trusses, it is generally best to compute the weight per lineal foot of the entire lateral system before the trusses are touched, because the dead load for the latter will be affected by the weight of the former.

Next it is necessary to assume the weight of metal per lineal foot for the trusses, using, if necessary, the formulæ given previously in this chapter. This completes the data for the preliminary dead load, which will consist of the following items :

First. Flooring (timber, track, pavement, etc.).

Second. Floor system (stringers, stringer-bracing, and floor-beams).

Third. Lateral system (upper and lower lateral systems, vertical sway-bracing, and portal-bracing).

Fourth. Trusses.

In making up the dead load, the end floor-beams and pedestals must not be included, as their weight produces no bending moment on the span.

The dead-load stresses in trusses are always found analytically for spans with parallel chords and equal panel lengths; but for all other cases they are determined graphically, and are checked by a single numerical calculation at the member where the graphics stop.

Whenever it is practicable, in making arithmetical computations, the slide-rule is employed. For ordinary work, in which the total stresses can be written with six figures, a twelve-inch slide-rule will give the stresses accurately in thousands of pounds; but where the stresses are greater, Thacher's cylindrical slide-rule is employed.

The live-load stresses are found by the method explained in Chapter XIX.

The computation of all stresses found analytically is facilitated by determining the trigonometrical functions involved in the calculations, and multiplying the panel loads by them. By setting these products on the slide-rule and using the proper tabulated coefficients, it is often practicable to read off a large series of stresses without resetting the slide.

The dead-load stresses and the live-load stresses are written on separate diagrams on the calculation-sheets.

The impact stresses are found from the live-load stresses by slide-rule from the formulæ given in either Chapter XIV or Chapter XVI, as the case may be, or from the corresponding tables at the end of the book, and are written on a separate diagram

Next are computed all the wind-stresses which could possibly affect the sizes of the sections of main-truss members, and these are recorded either on a separate diagram or on one of those already prepared, in the latter case care being taken to indicate that each such stress is marked as a wind-load stress.

Next the various combinations of all stresses are made and recorded on a new diagram, after which the required sections of all main members are figured according to the specifications, and are recorded on the same diagram; then the actual sections are proportioned and recorded there also.

The exact lengths of all members, including camber allowances, are next figured and recorded on the last-mentioned diagram.

Next the weight of metal in the trusses is estimated. For preliminary estimates, the weights of details are percentaged from recorded results of previous similar estimates; but if the structure be of an unusual type or size, the details are sketched and their weights are computed.

Next the total weight of metal in the structure is figured, and the dead load is checked. If it does not agree with that assumed within the limit of error set in the specifications, a new dead load is assumed, and the entire computations of total stresses, sections, and truss weights are made anew. It is very seldom, however, that it is necessary to make these calculations more than once, owing to the great mass of accumulated data concerning weights of metal in all kinds of bridges.

In making any set of calculations the computer should check back on his work at short intervals, so as to see that no error has been made, because the effects of such errors often extend over all succeeding computations.

In determining stresses graphically, the frame-diagram should be laid out on as large a scale as is convenient, and the load-diagram should be made as small as practicable; for the large frame gives great accuracy in inclinations of members, which is the all-important point in graphical computations, and the small load-diagram confines the graphics to a reasonable space. If the inclinations are correct, accurate results will be obtained with a very small load-diagram. The author's limits of error for graphical work are one quarter of one per cent at mid-span and one per cent at the far end of span. Should the error exceed these limits, the graphical work has to be done anew. Smooth paper, sharp pencils, true triangles, and perfect straight-edges are necessary to secure good results, to which list should be added painstaking accuracy in every manipulation of the appliances.

All calculations on the standard sheets are made in black copying-ink; and when they are checked by another computer, as is the invariable custom in the author's office, all check-marks and corrections are made in red ink, and each page checked is so marked and initialed by the checking computer, who not only verifies all the numerical calculations, but also follows carefully each step in the design so as to guard against all possible errors. The work of checking is greatly facilitated, if all the steps taken are indicated plainly, so that they can be easily followed by the checker. Each result checked is ticked off with red ink.

MAKING DRAWINGS.

Owing to the necessity for having several copies made of each drawing, the latter is first laid out in pencil on brown paper, and is copied in ink on tracing-cloth. In some simple designs, however, the pencilling is done directly on the tracing-cloth; but this is the exception rather than the rule. For convenience in handling and filing, it is very desirable to have all drawings made of a uniform size. After several years of experience, a size of twenty-nine inches in width and thirty-eight inches in length has been adopted as best suited for bridge plans. This size may be used for all detail drawings and stress-diagrams, but it is often necessary to increase the length for profiles and general drawings. The drawing is always made on the rough side of the tracing-cloth, as it is often convenient to do a considerable amount of drawing and writing in pencil on the sheet. Another reason for using the rough side is that any erasure shows less thereon than it would on the smooth side, and it is often necessary to do considerable erasing on tracings.

As before stated, the first drawings to be made are the general profile and plan with cross-sections, to establish all the main dimensions of the structure. These drawings can be prepared before the computations are finished. Next come the stress-diagrams, which should contain the cambered lengths of all members, the dead load, live load, impact and wind-load stresses, and the greatest combinations of same, the sections required and those used for each main member, and the following general data:

First. Length of span from centre to centre of end-pins.

Second. Number of panels.

Third. Perpendicular distance between central planes of trusses.

Fourth. Depths of trusses.

Fifth. Dead load for floor system per lineal foot of span.

Sixth. Dead load for trusses per lineal foot of span.

Seventh. Live load for stringers per lineal foot of span.

Eighth. Live load for floor-beams per lineal foot of span.

Ninth. Live load for trusses per lineal foot of span.

Tenth. Wind load on upper lateral system per lineal foot of span.

Eleventh. Wind load on lower lateral system per lineal foot of span.

Twelfth. Clearance required above base of rail or floor.

Thirteenth. Kinds of materials to be employed in all parts of structure.

Fourteenth. Diameters of rivets to be used.

The stress-diagram proper may be simply a line-drawing, each main member being represented by a single right line, or all the main members may be drawn to scale by means of their periphery-lines. The latter method is generally adopted because of the improved appearance of the sheet which it affords. The scale for any stress-diagram should be large enough to give plenty of room between panel points to contain all the necessary writing.

After the stress-diagrams are completed, the detail drawings are begun. There is considerable difference in the methods employed by consulting engineers to convey to manufacturers an understanding of the design which they desire to have executed in the shops. Some insist that the only proper method for the engineer to pursue, if he desires his details to be followed, is to make complete working or shop drawings, ready to be turned over to the template makers, while others prefer to make what are termed general detail drawings, which show to exact scale all the details, and give all important dimensions and the number of rivets in each connection, but which do not locate each rivet by figures, leaving the working drawings to be made by the manufacturer. When the latter method is adopted the working drawings must be sent in duplicate to the engineer for his approval before any of the work is sent into the shops, the said drawings being checked by the engineer's assistants, not only to see that they agree in every important particular with the original drawings, but also to make sure that they contain no errors of any kind.

The latter method is the one which the author invariably employs, and for adopting it he gives the following reasons:

First. Each bridge-shop has certain methods of doing work, which demand that the working drawings be made in accordance therewith; otherwise the cost of the manufacture is materially increased. These methods cannot be considered by the engineer, who has neither the time nor the inclination to go to the trouble of acquainting himself with the various methods of all the leading bridge-shops of the country.

Second. The nature of the work of a consulting engineer is not such as to justify him in keeping together enough trained draftsmen to execute with sufficient rapidity the large amount of drawing necessary, if the first-named method be followed.

Third. The capacity for accomplishing work in a consulting-engineer's office when the second method is employed is probably three times as great as it would be were the first method adopted.

Fourth. With the careful and thorough system of checking shop-drawings in vogue in the author's office, all the advantages to be gained by making complete working drawings are obtained by the much simpler method of making complete detail drawings.

Fifth. The manufacturer always appears to be better pleased and satisfied if the making of the shop-drawings be left to him; and the work of manufacturing the metal proceeds more smoothly in consequence.

In starting a detail drawing, the first thing to be done is to lay out a sheet of standard size. If the subject be a framed structure, such as a bridge or roof truss, it will greatly economize space on the drawing if the skeleton frame be laid out on a small scale, say three-eighths or one-half inch to the foot, thus giving the proper inclinations of all members, and if the details at all the panel points and connections be made to a larger scale, say three quarters of an inch or an inch to the foot. The centre-of-gravity lines of all main members should coincide with the lines of the skeleton diagram. For the details of ordinary bridges the scales just mentioned will be found the most satisfactory.

It is a very common error among bridge-draftsmen, when two different scales are used, to make the principal lines of the main members continuous between panel points, thus exaggerating the apparent size of the said members. This is entirely wrong, and is often the source of serious errors in the shops. In such drawings, the main members should be broken off before their principal lines meet midway between the panel points; and it is often advisable to show a section of the member between the broken ends.

After deciding upon the scales, the next step is to determine what portions of the structure are to be shown on each sheet, if more than one is to be made, and what is the best possible arrangement for all details on each sheet so as to fill it uniformly and allow ample space for illustrating each detail in the requisite number of views. For short spans, up to say two hundred feet, by carefully arranging the details, everything can be shown clearly on a standard sheet of twenty-nine inches by thirty-eight inches. The sizes of all connecting-plates, stay-plates, lacing-bars, connecting-angles, pins, fillers, rivets, etc., should be given, also those of all main members; and the exact spacing from back to back of all angles, channels, and webs, forming the various members, should be clearly indicated. The packing at all panel points should be shown, and the exact spacings therefor should be given by figures.

There should be indicated also all leading dimensions, such as the exact cambered lengths from centre to centre of pin-holes for all truss members; the vertical distance from centre of bottom-chord pins to base of rail; the vertical distance from centre of bottom-chord pins to bottom of floor-beams; the vertical distance from base of rail to top of masonry; the clearance required above base of rail; the spacing of anchor-bolts; the lengths of all built members beyond centres of pin-holes; the spacing of rivets in flanges of stringers, floor-beams, and chord members in a general way, such as "16 spaces of 3″ each," or "3″ spacing as nearly as may be"; the distance from back to back of opposite flange angles in all girders and struts; the widths of webs of all plate girders; the spacing of stiffening angles; etc., etc. All joints which are to be planed or faced should be so indicated.

Each sheet should have a general and descriptive title written in a neat but plain style of lettering. The title and the number of the drawing should be placed in the lower right-hand corner.

A single line drawn one-half inch from each edge of the sheet should define its margin, and if a rather fine line be drawn for each boundary of the tracing, and the sheet be trimmed just up to these boundary-lines, the blue-printer will have a well-defined border to which to trim his prints.

All lettering should be plain, but executed in a neat and workmanlike manner. Nothing adds more to the appearance of a drawing than neat lettering. Special care should be taken to locate all dimension-lines so there can be no doubt as to the distances they are intended to fix. All notes should be written in positions where they will be easily noticed, and so that they will not interfere with the lines of the drawing.

A set of general notes should be given on each sheet of details, specifying the kinds of material, the sizes of rivets, the diameters of rivet-holes before and after reaming, the manner in which all plates are to be finished, etc.

After each sheet is pencilled, it should be checked carefully to see that there are no errors thereon; then, after the tracing is finished, it must be checked in detail—if possible by some one who was not concerned in its preparation.

The following standard instructions of the author's to his office-assistants concerning the checking of drawings will indicate what such checking should accomplish and the essential thoroughness thereof:

GENERAL DETAIL DRAWINGS.

First. Go over all drawings for the entire design and see that every detail of the structure is shown in a sufficient number of views to make clear to the manufacturers exactly what is intended by the designer.

Second. See that every detail has been dimensioned so that it can be readily laid out on the working drawings. See also that all sections of connection angles, fillers, etc., are given.

Third. See that proper descriptive notes are given wherever necessary to make clear the reasons for any special details.

Fourth. Examine each detail and see that every portion of it is strong enough to carry properly the greatest stress that can ever come upon it. Make sure that enough rivets have been used, and that they are indicated to be countersunk or flattened wherever necessary to provide proper clearance.

Fifth. In checking up the packing at the panel points, see that all members which are to be brought on to the pin are shown, and that a sufficient clearance has been figured for each. Make sure that all forked ends have the requisite strength, and that diaphragms between same have been used wherever necessary. Check up the bearing of each member on the pin, and make sure that plenty of rivets have been used to convey the stress from the extension-plates to the main member. Remember that the stress to be provided for at the bottom of a vertical post is not the stress on the post itself, but the algebraic sum of the vertical components of the stresses in all diagonals attaching to the pin at the foot of post, or, approximately and on the side of safety, the stress on the post plus one half of a panel-floor load. See that no bar diverges from the central plane of truss more than one eighth ($\frac{1}{8}$) of an inch to the foot.

See that fillers are shown and their sizes given wherever they are necessary to hold the members to exact position on the pins.

Check all pins for the greatest bending moments coming on them, determining the same by combining the bending moments in two directions at right angles to each other.

Sixth. See that the centre-of-gravity lines of all members are shown; and where any such line is not in the central plane of member, see that it is located from the side of the section.

Seventh. Wherever a drawing is either wholly or partially shown in section, see that the exact point at which the section is taken is indicated in writing, and that the section line is properly shown on the other views to which the note refers.

Eighth. Compare all sections of members and all leading dimensions with those on the stress-diagram, and see that they correspond thereto.

Ninth. See that all stay-plates and lacing-bars are shown, and that the sizes for same are given; also that these sizes comply with the requirements of the specifications. The inclinations of all lacing-bars should be given.

Tenth. See that all extension-plates of forked ends are carried at least six inches inside the end stay-plates, and that they are strong enough to develop the full strength of the main member, even though the computed stress be small.

Eleventh. See that all reinforcing-plates at ends of members are so distributed as to balance as nearly as practicable the bearing on the two sides of the main section.

Twelfth. Compare drawings which show the same details, so as to make sure that all are alike.

Thirteenth. See that the same style of detailing has been followed on all drawings. Where several draftsmen are employed on the same piece of work there is liable to be quite a diversity of details, illustrating the individualities of the various draftsmen making them.

Fourteenth. When a change is made in any part of a drawing, see that said change is carried through all the sheets which are affected thereby.

Fifteenth. See that when any drawing or portion thereof is abandoned it is so indicated clearly throughout all the drawings.

Sixteenth. Check all forked ends for transverse bending, and see that they have been reinforced wherever necessary.

Seventeenth. Wherever timber-bolts are to be used, see that they are plainly indicated, that their sizes and lengths are given, and that washers are provided beneath all heads where the bearing is on the wood.

Eighteenth. See that all screw-ends of rods are upset, unless they are to have cold-pressed threads. See that all diagonal rods are provided with proper adjustments, and that all clevis-pins and plates are of proper strength. See that no pins of less than two and a half inches diameter are used, and that they are set at least one and one-half diameters from edge of plate.

Nineteenth. See that each sheet is provided with general notes, as follows.

A. Kinds of material to be used throughout the structure.

B. Diameters for rivets.

C. Sizes of rivet-holes before and after reaming.

D. Manner in which the edges of all web-plates are to be finished.

E. What ends are to be faced and what are not.

Twentieth. See that all notes are written in good English, that all words are spelled correctly, and that they express exactly what is intended.

Twenty-first. See that each drawing is provided with proper titles, that it is numbered correctly, that the scale or scales are indicated, and that the name of the draftsman and date of completion of drawing are given.

Twenty second. See that the drawings scale, and, if they do not, make a note saying that the dimensions written on the drawings are to be followed in preference to the scale where there is any discrepancy between the two.

Twenty-third. In short, check over all details, dimensions, sections, and notes given on the drawings, so as to make sure that everything is in strict accordance with the specifications and with the data furnished for the structure.

SHOP DRAWINGS.

First. Make sure that the sections and details conform in every particular with those given on the general detail drawings and stress-diagrams, excepting in minor points, where slight changes may be made to facilitate the work in the shops, provided, of course, that such alterations do not in any way impair the strength, durability, or appearance.

Second. Check over all field connections to see that there are no rivets which are so located that they cannot be satisfactorily driven in the field.

Third. See that all members have proper clearances at panel points, and that all rivet-heads, wherever necessary to provide such clearances, are countersunk or flattened.

Fourth. Check over all lengths of members and rivet-spacing for field connections to make sure that the holes will match in the field.

Fifth. Check over all bills of material to see that the proper number of pieces have been ordered, and that they are of proper sections and lengths.

Sixth. Always have the shop-drawings sent to the office in duplicate, and check up the two sets, retaining one set in the office and returning the other set with corrections or approval marked thereon. Where drawings are returned to shops with corrections marked on them, revised prints must be sent for approval before the work is put into the shops.

It is often necessary to make changes on a tracing, and in doing so great care should be exercised, otherwise a drawing which has cost considerable time and money may be ruined.

For making slight erasures, a very sharp knife skilfully used will be found effective, as it can be so manipulated as to affect nothing but the parts to be erased. Another expedient, where only a slight erasure is to be made, is to use a thin sheet of celluloid or durable cardboard, in which are cut small holes corresponding to the work to be changed. This sheet is laid on the drawing so that a hole comes over the part to be erased, then a sand-eraser is rubbed over the hole, and nothing is damaged except the portion which is changed.

FILING DRAWINGS, CALCULATIONS, SPECIFICATIONS, ETC.

In the course of a few years' practice the office records of a consulting engineer grow to such proportions that, unless some systematic method of filing and indexing them be adopted, it is impossible to refer thereto without a great deal of delay and annoyance. The filing of calculations and specifications is a comparatively easy matter, but to keep an accumulating lot of drawings in good shape for ready reference is by no means such. During the time that the author has been engaged in active practice several methods have been employed for filing tracings. One great difficulty with the earlier drawings was that they were of varying dimensions, some as large as forty-two inches by ninety-six inches, and others belonging to the same set as small as eighteen inches square. At first large cases of drawers were used for laying out the tracings flat, each tracing being stamped with numbers designating the lot and drawer to which it belonged, and an index being kept of all drawings recording the numbers of the lot and drawer. The objections to this method were that the smaller drawings got lost among the larger ones, thus often necessitating a complete overhauling of an entire drawer to find a tracing, and it was impossible to keep the large drawings from becoming folded and cracked at the edges and corners.

Later it was deemed advisable to bind each set of drawings together with patent fasteners along one end, but this method was soon abandoned, owing to the difficulty encountered in getting out tracings for blue-printing and reference.

The method of laying the tracings flat in drawers was abandoned, and they are now filed in cardboard tubes, thirty inches long and four inches in diameter, with tightly fitting covers of the same material. Each tube has on the cover its index number and a type-written list of the tracings it contains. The tracings are rolled in small bunches of four or five, and each bunch is held to small diameter with a rubber band, to which is attached a tag giving the number and title of each of the sheets contained in the roll. Five rolls are placed in each tube, making a total of from twenty to twenty-five tracings per tube. The tracings should not be rolled so closely that they will become creased.

An index is kept of all tubes, giving their numbers and the titles of the drawings contained in each; and there is in addition an alphabetical index of the drawings.

The tubes are set in cases with their covers exposed, and are so arranged that any tube can be easily reached or removed from the case if necessary.

Copies of all shop-drawings are also kept on file for reference. These are put in larger tubes, and as there is never any necessity for separating a set, as is the case with the tracings, each set is bound together when complete. The shop-drawings are all included in the two indices.

The specifications and calculations are kept in filing cases prepared especially for them, and both are indexed. These cases consist of a series of small shelves about one and a half inches apart, each shelf being numbered.

When a set of calculations is complete, the sheets are all bound together in one book with removable fastenings, so that they can be easily separated when it is necessary to distribute them among several draftsmen. These sets are all numbered with the numbers of the shelves on which they are to be filed.

In indexing all work every article should be indexed under as many headings as practicable.

OFFICE MATERIALS.

All calculation-blanks should be of an extra-good quality of paper, capable of withstanding a great deal of erasing and scratching, which is often necessary in making sketches for details. The tracing-cloth should be of the best quality, as it is impracticable to make a good drawing on poor cloth. The best brand that the author has ever used is the Imperial.

Powdered chalk or talcum should be rubbed over the surface of the tracing-cloth to make it take the ink uniformly. Pencil-marks and dirt can be easily removed from a tracing by moistening a towel in benzine and washing the surface of the cloth with it. If a good quality of ink be used it will not be affected by such washing.

There are many liquid India inks in the market, but none of them will give quite as good results as will the genuine stick ink when properly ground; nevertheless, except for very fine work, the former are preferable on account of the saving of time which they effect. Higgins' water-proof ink is the most satisfactory which has yet been tried in the author's office.

A good quality of brown detail paper is very essential, for there is in all kinds of detailing a great deal of erasing to be done; and time is always saved by using good, tough paper that does not rough up by having an eraser used upon it.

CONCLUSION.

In concluding this chapter on "Office Practice" the author desires to again call the reader's attention to the necessity for adopting the most systematic methods possible for doing all kinds of work, keeping all kinds of records, and filing all kinds of accumulated material. As soon as a large piece of work is finished a thorough systemization should be made of the knowledge obtained in making both the design and the various calculations, so that the office force shall be able to use the same to the best possible advantage when starting on another similar piece of work. And whenever there is any spare time in the office for any of the employees it should be devoted to accumulating, digesting, and putting in convenient form for use the results of previous investigations, and to doing such work as tabulating and recording on diagrams the weights of metal per lineal foot of span for bridges of all kinds.

Finally, in bringing this little treatise to a close the author feels that he cannot do better than to repeat from Chapter II the following principle: "The systemization of all that one does in connection with his professional work is one of the most important steps that can be taken towards the attainment of success."

TABLES.

Table I.

COEFFICIENTS OF IMPACT FOR RAILWAY BRIDGES.

I = Impact.
L = Length of Span in Feet.

$$I = \frac{400}{L + 500}.$$

L	I	L	I	L	I
1	0.7984	50	0.7273	99	0.6678
2	0.7968	51	0.7260	100	0.6667
3	0.7952	52	0.7247	105	0.6612
4	0.7936	53	0.7233	110	0.6557
5	0.7921	54	0.7220	115	0.6504
6	0.7905	55	0.7207	120	0.6452
7	0.7889	56	0.7194	125	0.6400
8	0.7874	57	0.7181	130	0.6349
9	0.7858	58	0.7169	135	0.6299
10	0.7843	59	0.7156	140	0.6250
11	0.7828	60	0.7143	145	0.6202
12	0.7812	61	0.7130	150	0.6154
13	0.7797	62	0.7117	155	0.6107
14	0.7782	63	0.7105	160	0.6061
15	0.7767	64	0.7092	165	0.6015
16	0.7752	65	0.7080	170	0.5970
17	0.7737	66	0.7067	175	0.5926
18	0.7722	67	0.7055	180	0.5882
19	0.7707	68	0.7042	185	0.5839
20	0.7692	69	0.7030	190	0.5797
21	0.7678	70	0.7018	195	0.5755
22	0.7663	71	0.7005	200	0.5714
23	0.7648	72	0.6993	210	0.5634
24	0.7634	73	0.6981	220	0.5556
25	0.7619	74	0 6968	230	0.5480
26	0.7605	75	0.6957	240	0.5405
27	0.7590	76	0.6944	250	0.5333
28	0.7576	77	0.6932	260	0.5263
29	0.7561	78	0.6920	270	0.5195
30	0.7547	79	0.6908	280	0.5128
31	0.7533	80	0.6897	290	0.5063
32	0.7519	81	0.6885	300	0.5000
33	0.7505	82	0.6873	325	0.4848
34	0.7491	83	0.6861	350	0.4706
35	0.7477	84	0.6849	375	9.4571
36	0.7463	85	0.6838	400	0.4444
37	0.7449	86	0.6826	450	0.4211
38	0.7435	87	0.6814	500	0.4000
39	0.7421	88	0.6803	550	0.3810
40	0.7407	89	0.6791	600	0.3636
41	0.7394	90	0.6780	650	0.3478
42	0.7380	91	0.6768	700	0.3333
43	0.7367	92	0.6757	750	0.3200
44	0.7353	93	0.6745	800	0.3077
45	0.7339	94	0.6734	850	0.2963
46	0.7326	95	0.6723	900	0.2857
47	0.7313	96	0.6711	950	0.2759
48	0.7299	97	0.6700	1000	0.2667
49	0.7286	98	0.6689		

Table II.

COEFFICIENTS OF IMPACT FOR HIGHWAY BRIDGES.

I = Impact.
L = Length of Span in Feet.

$$I = \frac{100}{L + 150}.$$

L	I	L	I	L	I
1	0.6623	50	0.5000	99	0.4016
2	0.6579	51	0.4975	100	0.4000
3	0.6536	52	0.4951	105	0.3922
4	0.6494	53	0.4926	110	0.3846
5	0.6452	54	0.4902	115	0.3774
6	0.6410	55	0.4878	120	0.3704
7	0.6369	56	0.4854	125	0.3636
8	0.6329	57	0.4831	130	0.3571
9	0.6289	58	0.4808	135	0.3509
10	0.6250	59	0.4785	140	0.3448
11	0.6211	60	0.4762	145	0.3389
12	0.6173	61	0.4739	150	0.3333
13	0.6134	62	0.4717	155	0.3279
14	0.6098	63	0.4695	160	0.3226
15	0.6061	64	0.4673	165	0.3175
16	0.6024	65	0.4651	170	0.3125
17	0.5988	66	0.4630	175	0.3077
18	0.5952	67	0.4608	180	0.3030
19	0.5917	68	0.4587	185	0.2985
20	0.5882	69	0.4566	190	0.2941
21	0.5848	70	0.4546	195	0.2899
22	0.5814	71	0.4525	200	0.2857
23	0.5780	72	0.4505	210	0.2778
24	0.5747	73	0.4484	220	0.2703
25	0.5714	74	0.4464	230	0.2632
26	0.5682	75	0.4444	240	0.2564
27	0.5650	76	0.4425	250	0.2500
28	0.5618	77	0.4405	260	0.2439
29	0.5586	78	0.4386	270	0.2381
30	0.5556	79	0.4367	280	0.2326
31	0.5525	80	0.4348	290	0.2273
32	0.5495	81	0.4329	300	0.2222
33	0.5465	82	0.4310	325	0.2105
34	0.5435	83	0.4292	350	0.2000
35	0.5405	84	0.4274	375	0.1905
36	0.5376	85	0.4255	400	0.1818
37	0.5348	86	0.4237	450	0.1667
38	0.5319	87	0.4219	500	0.1539
39	0.5291	88	0.4202	550	0.1429
40	0.5263	89	0.4184	600	0.1333
41	0.5236	90	0.4167	650	0.1250
42	0.5208	91	0.4149	700	0.1176
43	0.5181	92	0.4131	750	0.1111
44	0.5155	93	0.4115	800	0.1053
45	0.5128	94	0.4098	850	0.1000
46	0.5102	95	0.4082	900	0.0952
47	0.5076	96	0.4065	950	0.0909
48	0.5051	97	0.4049	1000	0.0870
49	0.5025	98	0.4033		

TABLE III.

INTENSITIES FOR INCLINED END POSTS.

$$P = 18{,}000 - 80\frac{l}{r}.$$

$\frac{l}{r}$	P	$\frac{l}{r}$	P	$\frac{l}{r}$	P	$\frac{l}{r}$	P	$\frac{l}{r}$	P
1	17920	25	16000	49	14080	73	12160	97	10240
2	17840	26	15920	50	14000	74	12080	98	10160
3	17760	27	15840	51	13920	75	12000	99	10080
4	17680	28	15760	52	13840	76	11920	100	10000
5	17600	29	15680	53	13760	77	11840	101	9920
6	17520	30	15600	54	13680	78	11760	102	9840
7	17440	31	15520	55	13600	79	11680	103	9760
8	17360	32	15440	56	13520	80	11600	104	9680
9	17280	33	15360	57	13440	81	11520	105	9600
10	17200	34	15280	58	13360	82	11440	106	9520
11	17120	35	15200	59	13280	83	11360	107	9440
12	17040	36	15120	60	13200	84	11280	108	9360
13	16960	37	15040	61	13120	85	11200	109	9280
14	16880	38	14960	62	13040	86	11120	110	9200
15	16800	39	14880	63	12960	87	11040	111	9120
16	16720	40	14800	64	12880	88	10960	112	9040
17	16640	41	14720	65	12800	89	10880	113	8960
18	16560	42	14640	66	12720	90	10800	114	8880
19	16480	43	14560	67	12640	91	10720	115	8800
20	16400	44	14480	68	12560	92	10640	116	8720
21	16320	45	14400	69	12480	93	10560	117	8640
22	16240	46	14320	70	12400	94	10480	118	8560
23	16160	47	14240	71	12320	95	10400	119	8480
24	16080	48	14160	72	12240	96	10320	120	8400

Table IV.

INTENSITIES FOR TOP-CHORD COMPRESSION-MEMBERS.

$$P = 18{,}000 - 70\frac{l}{r}.$$

$\frac{l}{r}$	P	$\frac{l}{r}$	P	$\frac{l}{r}$	P	$\frac{l}{r}$	P	$\frac{l}{r}$	P
1	17930	25	16250	49	14570	73	12890	97	11210
2	17860	26	16180	50	14500	74	12820	98	11140
3	17790	27	16110	51	14430	75	12750	99	11070
4	17720	28	16040	52	14360	76	12680	100	11000
5	17650	29	15970	53	14290	77	12610	101	10930
6	17580	30	15900	54	14220	78	12540	102	10860
7	17510	31	15830	55	14150	79	12470	103	10790
8	17440	32	15760	56	14080	80	12400	104	10720
9	17370	33	15690	57	14010	81	12330	105	10650
10	17300	34	15620	58	13940	82	12260	106	10580
11	17230	35	15550	59	13870	83	12190	107	10510
12	17160	36	15480	60	13800	84	12120	108	10440
13	17090	37	15410	61	13730	85	12050	109	10370
14	17020	38	15340	62	13660	86	11980	110	10300
15	16950	39	15270	63	13590	87	11910	111	10230
16	16880	40	15200	64	13520	88	11840	112	10160
17	16810	41	15130	65	13450	89	11770	113	10090
18	16740	42	15060	66	13380	90	11700	114	10020
19	15670	43	14990	67	13310	91	11630	115	9950
20	16600	44	14920	68	13240	92	11560	116	9880
21	16530	45	14850	69	13170	93	11490	117	9810
22	16460	46	14780	70	13100	94	11420	118	9740
23	16390	47	14710	71	13030	95	11350	119	9670
24	16320	48	14640	72	12960	96	11280	120	9600

TABLE V.

INTENSITIES FOR INTERMEDIATE POSTS AND SUB-DIAGONALS.

$$P = 16{,}000 - 80\frac{l}{r}.$$

$\frac{l}{r}$	P	$\frac{l}{r}$	P	$\frac{l}{r}$	P	$\frac{l}{r}$	P	$\frac{l}{r}$	P
1	15920	25	14000	49	12080	73	10160	97	8240
2	15840	26	13920	50	12000	74	10080	98	8160
3	15760	27	13840	51	11920	75	10000	99	8080
4	15680	28	13760	52	11840	76	9920	100	8000
5	15600	29	13680	53	11760	77	9840	101	7920
6	15520	30	13600	54	11680	78	9760	102	7840
7	15440	31	13520	55	11600	79	9680	103	7760
8	15360	32	13440	56	11520	80	9600	104	7680
9	15280	33	13360	57	11440	81	9520	105	7600
10	15200	34	13280	58	11360	82	9440	106	7520
11	15120	35	13200	59	11280	83	9360	107	7440
12	15040	36	13120	60	11200	84	9280	108	7360
13	14960	37	13040	61	11120	85	9200	109	7280
14	14880	38	12960	62	11040	86	9120	110	7200
15	14800	39	12880	63	10960	87	9040	111	7120
16	14720	40	12800	64	10880	88	8960	112	7040
17	14640	41	12720	65	10800	89	8880	113	6960
18	14560	42	12640	66	10720	90	8800	114	6880
19	14480	43	12560	67	10640	91	8720	115	6800
20	14400	44	12480	68	10560	92	8640	116	6720
21	14320	45	12400	69	10480	93	8560	117	6640
22	14240	46	12320	70	10400	94	8480	118	6560
23	14160	47	12240	71	10320	95	8400	119	6480
24	14080	48	12160	72	10240	96	8320	120	6400

Table VI.

INTENSITIES FOR COLUMNS OF VIADUCTS AND ELEVATED RAILROADS AND FOR ALL LATERAL STRUTS.

$$P = 16{,}000 - 60\frac{l}{r}.$$

$\frac{l}{r}$	P	$\frac{l}{r}$	P	$\frac{l}{r}$	P	$\frac{l}{r}$	P	$\frac{l}{r}$	P	$\frac{l}{r}$	P
1	15940	26	14440	51	12940	76	11440	101	9940	126	8440
2	15880	27	14380	52	12880	77	11380	102	9880	127	8380
3	15820	28	14320	53	12820	78	11320	103	9820	128	8320
4	15760	29	14260	54	12760	79	11260	104	9760	129	8260
5	15700	30	14200	55	12700	80	11200	105	9700	130	8200
6	15640	31	14140	56	12640	81	11140	106	9640	131	8140
7	15580	32	14080	57	12580	82	11080	107	9580	132	8080
8	15520	33	14020	58	12520	83	11020	108	9520	133	8020
9	15460	34	13960	59	12460	84	10960	109	9460	134	7960
10	15400	35	13900	60	12400	85	10900	110	9400	135	7900
11	15340	36	13840	61	12340	86	10840	111	9340	136	7840
12	15280	37	13780	62	12280	87	10780	112	9280	137	7780
13	15220	38	13720	63	12220	88	10720	113	9220	138	7720
14	15160	39	13660	64	12160	89	10660	114	9160	139	7660
15	15100	40	13600	65	12100	90	10600	115	9100	140	7600
16	15040	41	13540	66	12040	91	10540	116	9040	141	7540
17	14980	42	13480	67	11980	92	10480	117	8980	142	7480
18	14920	43	13420	68	11920	93	10420	118	8920	143	7420
19	14860	44	13360	69	11860	94	10360	119	8860	144	7360
20	14800	45	13300	70	11800	95	10300	120	8800	145	7300
21	14740	46	13240	71	11740	96	10240	121	8740	146	7240
22	14680	47	13180	72	11680	97	10180	122	8680	147	7180
23	14620	48	13120	73	11620	98	10120	123	8620	148	7120
24	14560	49	13060	74	11560	99	10060	124	8560	149	7060
25	14500	50	13000	75	11500	100	10000	125	8500	150	7000

TABLE VII.

CENTRIFUGAL FORCE IN PERCENTAGES OF LIVE LOAD.

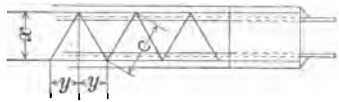
$C.F. = \frac{V^2 \times 100}{32.2 \times R}$. V = Velocity in feet per second. R = Radius in feet.

Degree.	Velocity in Miles per Hour.								
	10	15	20	25	30	35	40	50	60
1	0.12	0.26	0.46	0.73	1.05	1.43	1.87	2.91	4.20
2	0.23	0.53	0.93	1.46	2.10	2.86	3.73	5.82	8.40
3	0.35	0.79	1.40	2.19	3.15	4.28	5.60	8.74	12.59
4	0.47	1.05	1.86	2.92	4.20	5.71	7.46	11.65	16.78
5	0.58	1.31	2.33	3.65	5.25	7.14	9.33	14.57	20.99
6	0.70	1.57	2.79	4.37	6.30	8.56	11.19	17.48	25.17
7	0.82	1.84	3.26	5.10	7.34	9.99	13.05	20.39	29.36
8	0.93	2.10	3.73	5.82	8.39	11.42	14.91	23.30	33.55
9	1.05	2.36	4.19	6.55	9.43	12.84	16.77	26.20	37.74
10	1.16	2.62	4.66	7.28	10.48	14.26	18.63	29.11	41.92
11	1.28	2.89	5.12	8.00	11.52	15.68	20.40	32.01	
12	1.40	3.14	5.58	8.73	12.59	17.11	22.35	34.89	
13	1.51	3.40	6.05	9.45	13.61	18.53	24.20		
14	1.63	3.66	6.51	10.17	14.65	19.94	26.05		
15	1.74	3.92	6.97	10.90	15.70	21.36	27.90		
16	1.86	4.18	7.43	11.62	16.73	22.77			
17	1.98	4.44	7.90	12.34	17.77	24.19			
18	2.09	4.70	8.36	13.06	18.81	25.60			
19	2.21	4.96	8.82	13.79	19.85	27.00			
20	2.32	5.22	9.28	14.51	20.88	28.42			
21	2.44	5.48	9.74	15.22	21.91	29.82			
22	2.55	5.74	10.20	15.94	22.95	31.23			
23	2.66	5.99	10.65	16.65	23.97	32.63			
24	2.78	6.25	11.11	17.37	25.00	34.02			
25	2.89	6.51	11.57	18.08	26.02	35.42			
26	3.01	6.76	12.02	18.79	27.05	36.81			
27	3.12	7.02	12.47	19.50	28.07	38.20			
28	3.23	7.27	12.93	20.20	29.09	39.59			
29	3.34	7.53	13.37	20.91	30.11	40.97			
30	3.46	7.78	13.83	21.62	31.12	42.35			
31	3.57	8.03	14.28	22.32	32.13	43.73			
32	3.68	8.29	14.73	23.03	33.15	45.11			
33	3.80	8.54	15.18	23.72	34.16	46.49			
34	3.91	8.79	15.62	24.42	35.16	47.85			
35	4.02	9.04	16.07	25.12	36.15	49.20			
36	4.13	9.29	16.51	25.81	37.16	50.57			
37	4.24	9.54	16.95	26.50	38.15	51.92			
38	4.35	9.79	17.39	27.19	39.14	53.27			
39	4.46	10.04	17.84	27.88	40.14	54.62			
40	4.57	10.28	18.27	28.57	41.12	55.97			
41	4.68	10.53	18.71	29.24	42.10	57.30			
42	4.79	10.77	19.15	29.93	43.09	58.66			
43	4.90	11.02	19.58	30.61	44.08	59.99			
44	5.01	11.26	20.01	31.29	45.04	61.29			
45	5.11	11.50	20.44	31.95	46.00	62.61			
46	5.22	11.74	20.87	32.63	46.97				
47	5.33	11.99	21.30	33.30	47.95				
48	5.44	12.23	21.73	33.98	48.92				
49	5.54	12.46	22.15	34.63	49.85				
50	5.65	12.71	22.58	35.30	50.82				

NOTE.—The stepped line shows the limiting percentages for a superelevation of 4″ for outer rail.

TABLE VIII.

SIZES AND WEIGHTS OF STAY-PLATES AND LACING-BARS FOR ORDINARY POSTS.

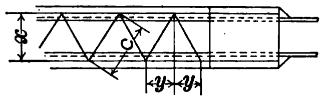


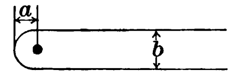
$$a=\frac{b}{2}+\frac{1''}{4}$$

Depth of Channel.	x	y	C	Size of Lacing-bar.	Size of Stay-plate.	Weight of Lacing in pounds per Ft. of Col.	Weight of One Stay-plate in pounds.	Size of Rivet.
6″	5″	3″	$5\frac{27}{32}$″	$1\frac{3}{4}$″ × $\frac{3}{8}$″	$6\frac{7}{8}$″ × $\frac{3}{8}$″ × 9″	13.22	6.58	⅝″
	6	4	$7\frac{7}{32}$	" "	$7\frac{7}{8}$ × $\frac{3}{8}$ × 9	11.45	7.53	
	7	4	$8\frac{1}{16}$	" "	$8\frac{7}{8}$ × $\frac{3}{8}$ × 12	12.39	11.32	
	8	$4\frac{1}{2}$	$9\frac{3}{16}$	" "	$9\frac{7}{8}$ × $\frac{3}{8}$ × 12	12.13	12.60	
	9	$5\frac{1}{4}$	$10\frac{13}{32}$	" "	$10\frac{7}{8}$ × $\frac{3}{8}$ × 15	11.42	17.34	
	10	$5\frac{3}{4}$	$11\frac{17}{32}$	" "	$11\frac{7}{8}$ × $\frac{3}{8}$ × 15	11.31	18.93	
7″	5	3	$5\frac{27}{32}$	" "	7 × $\frac{3}{8}$ × 9	13.22	6.70	
	6	4	$7\frac{7}{32}$	" "	8 × $\frac{3}{8}$ × 9	11.45	7.65	
	7	4	$8\frac{1}{16}$	" "	9 × $\frac{3}{8}$ × 12	12.39	11.48	
	8	$4\frac{1}{2}$	$9\frac{3}{16}$	" "	10 × $\frac{3}{8}$ × 12	12.13	12.75	
	9	$5\frac{1}{4}$	$10\frac{13}{32}$	" "	11 × $\frac{3}{8}$ × 15	11.42	17.54	
	10	$5\frac{3}{4}$	$11\frac{17}{32}$	" "	12 × $\frac{3}{8}$ × 15	11.31	19.13	
8″	6	4	$7\frac{7}{32}$	2″ × $\frac{3}{8}$″	8 × $\frac{3}{8}$ × 9	13.45	7.65	¾″
	7	4	$8\frac{1}{16}$	" "	9 × $\frac{3}{8}$ × 12	14.52	11.48	
	8	$4\frac{1}{2}$	$9\frac{3}{16}$	" "	10 × $\frac{3}{8}$ × 12	14.19	12.75	
	9	$5\frac{1}{4}$	$10\frac{13}{32}$	" "	11 × $\frac{3}{8}$ × 15	13.34	17.54	
	10	$5\frac{3}{4}$	$11\frac{17}{32}$	" "	12 × $\frac{3}{8}$ × 15	13.18	19.13	
	11	$6\frac{1}{4}$	$12\frac{21}{32}$	" "	13 × $\frac{3}{8}$ × 18	13.05	24.87	
9″	7	4	$8\frac{1}{16}$	" "	$9\frac{1}{2}$ × $\frac{3}{8}$ × 12	12.39	11.64	

TABLE VIII—*Continued.*

SIZES AND WEIGHTS OF STAY-PLATES AND LACING-BARS FOR ORDINARY POSTS.



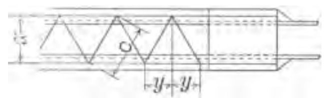


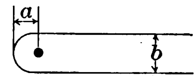
$$a=\frac{b}{2}+\frac{1''}{4}$$

Depth of Channel.	x	y	C	Size of Lacing-bar.	Size of Stay-plate.	Weight of Lacing in pounds per Ft. of Col.	Weight of One Stay-plate in pounds.	Size of Rivet.
9″	8″	4½″	9 3/16″	2″ × ⅜″	10⅛″ × ⅜″ × 12″	14.19	12.91	¾″
	9	5¼	10 13/32	" "	11⅛ × ⅜ × 15	13.31	17.76	
	10	5¾	11 17/32	" "	12⅛ × ⅜ × 15	13.18	19.32	
	11	6¼	12 21/32	" "	13⅛ × ⅜ × 18	13.05	25.11	
	12	7	13 29/32	" "	14⅛ × ⅜ × 18	12.56	27.02	
10″	8	4½	9 3/16	2¼″ × ⅜″	10¼ × ⅜ × 12	16.03	13.07	
	9	5¼	10 13/32	" "	11¼ × ⅜ × 15	15.53	17.94	
	10	5¾	11 17/32	" "	12¼ × ⅜ × 15	14.98	19.53	
	11	6¼	12 21/32	" "	13¼ × ⅜ × 18	14.74	25.35	
	12	7	13 29/32	" "	14¼ × ⅜ × 18	14.19	27.26	
	13	7½	15	" "	15¼ × ⅜ × 21	14.07	34.04	
	14	8	16⅛	" "	16¼ × ⅜ × 21	14.00	36.26	
	15	8¾	17⅜	" "	17¼ × ⅜ × 24	13.62	44.00	
12″	8	4½	9 3/16	" "	10⅝ × ⅜ × 12	16.03	13.55	
	9	5¼	10 13/32	" "	11⅝ × ⅜ × 15	15.53	18.55	
	10	5¾	11 17/32	" "	12⅝ × ⅜ × 15	14.98	20.13	
	11	6¼	12 21/32	" "	13⅝ × ⅜ × 18	14.74	26.06	
	12	7	13 29/32	" "	14⅝ × ⅜ × 18	14.19	27.98	
	13	7½	15	" "	15⅝ × ⅜ × 21	14.07	34.88	

Table VIII—*Continued.*

SIZES AND WEIGHTS OF STAY-PLATES AND LACING-BARS FOR ORDINARY POSTS.





$$a=\frac{b}{2}+\frac{1}{4}''$$

Depth of Channel.	x	y	C	Size of Lacing-bar.	Size of Stay-plate.	Weight of Lacing in pounds per Ft. of Col.	Weight of One Stay-plate in pounds.	Size of Rivet.
12″	14″	8″	16⅛″	2¼″ × ⅜″	16⅝″ × ⅜″ × 21″	14.00	37.10	¾″
	15	8¼	17⅜	" "	17⅝ × ⅜ × 24	13.62	44.97	
	16	9¼	18⅝	" "	18⅝ × ⅜ × 24	13.30	47.50	
15″	9	5¼	10 13/32	2½ × ⅜	12 × ⅜ × 15	17.66	19.13	⅞″
	10	5¾	11 17/32	" "	13 × ⅜ × 15	17.38	20.73	
	11	6¼	1′ 0 21/32	" "	14 × ⅜ × 18	17.14	26.79	
	12	7	1 1 29/32	" "	15 × ⅜ × 18	16.44	28.71	
	13	7¼	1 3	" "	16 × ⅜ × 21	16.27	35.70	
	14	8	1 4⅛	" "	17 × ⅜ × 21	16.15	37.94	
	15	8¾	1 5⅜	" "	18 × ⅜ × 24	15.68	45.92	
	16	9½	1 6⅝	" "	19 × ⅜ × 24	15.28	48.45	
	18	10¼	1 8 27/32	3 × 7/16	21 × ⅜ × 27	21.15	60.25	
	20	11¼	1 11 1/16	" ½	23 × 7/16 × 30	23.92	85.60	
	22	12¼	2 1 7/16	" 9/16	25 × 7/16 × 33	26.31	102.30	
	24	13¼	2 3 21/32	" "	27 × ½ × 36	26.25	137.75	
	26	15	2 6 1/32	1 ∠ 3″ × 2½″ × ⅜″	29 × 9/16 × 39		180.18	
	28	16¼	2 8⅜	" "	31 × 9/16 × 42		207.48	
	30	17¼	2 10 19/32	" "	33 × ⅝ × 45		263.03	

Table IX.

BENDING MOMENTS ON PINS.

Diam. of Pin.	Moments in In.-lbs. for Fibre Stress of		Diam. of Pin.	Moments in In.-lbs. for Fibre Stress of		Diam. of Pin.	Moments in In.-lbs. for Fibre Stress of	
	27,000 lbs. per sq. in.	35,100 lbs. per sq. in.		27,000 lbs. per sq. in.	35,100 lbs. per sq. in.		27,000 lbs. per sq. in.	35,100 lbs. per sq. in.
2″	21200	27570	6⅞″	861300	1119700	11⅝″	4164500	5413900
2⅛	25500	33200	7	909100	1181800	11¾	4300000	5590000
2¼	30200	39300	7⅛	958800	1246400	11⅞	4439300	5771100
2⅜	35500	46200	7¼	1010100	1313100	12	4580500	5954700
2½	41400	53800	7⅜	1063300	1382300	12⅛	4725000	6142500
2⅝	47900	62300	7½	1118300	1453800	12¼	4872700	6334500
2¾	55100	71700	7⅝	1175100	1527500	12⅜	5023600	6530700
2⅞	63000	81900	7¾	1233900	1604100	12½	5177200	6730400
3	71600	93100	7⅞	1294500	1682700	12⅝	5334700	6935100
3⅛	80800	105000	8	1357200	1764400	12¾	5494000	7142200
3¼	91000	118300	8⅛	1421800	1848300	12⅞	5657300	7354500
3⅜	101900	132500	8¼	1488500	1935100	13	5823600	7670700
3½	113600	147700	8⅜	1557000	2024100	13⅛	5993200	7791200
3⅝	126300	164200	8½	1627800	2116100	13¼	6166000	8015800
3¾	139800	181700	8⅝	1700700	2210900	13⅜	6342300	8245000
3⅞	154200	200500	8¾	1775700	2308400	13½	6521700	8478200
4	169600	220500	8⅞	1853000	2408900	13⅝	6704600	8716000
4⅛	186100	241900	9	1932300	2512100	13¾	6890900	8958200
4¼	203500	264500	9⅛	2013900	2618100	13⅞	7080500	9204700
4⅜	222000	288600	9¼	2097900	2727300	14	7273800	9455900
4½	241500	314000	9⅜	2184300	2839600	14⅛	7470000	9711000
4⅝	262200	340900	9½	2272900	2954800	14¼	7670700	9971900
4¾	284100	369300	9⅝	2363600	3072700	14⅜	7874000	10236200
4⅞	307100	399200	9¾	2457000	3194100	14½	8081100	10505400
5	331300	430700	9⅞	2552500	3318300	14⅝	8292000	10779600
5⅛	356800	463900	10	2650900	3446200	14¾	8506400	11058300
5¼	383600	498700	10⅛	2751300	3576700	14⅞	8724500	11341900
5⅜	411600	535100	10¼	2854400	3710700	15	8946200	11630100
5½	441000	573300	10⅜	2960300	3848400	15½	9871200	12832600
5⅝	471800	613300	10½	3068600	3989200	16	10857200	14114400
5¾	504000	655200	10⅝	3179500	4133400	16½	11907300	15479500
5⅞	537500	698700	10¾	3292900	4280800	17	13022900	16929800
6	572500	744300	10⅞	3409600	4432500	17½	14206300	18468200
6⅛	609200	792000	11	3528100	4586500	18	15459100	20096800
6¼	647100	841200	11⅛	3649900	4744900	18½	16783500	21818600
6⅜	686700	892700	11¼	3774300	4906600	19	18181300	23635700
6½	728000	946400	11⅜	3901500	5072000	19½	19654600	25551000
6⅝	770700	1001900	11½	4031400	5240800	20	21205800	27567500
6¾	815300	1059800						

Note.—27000 lbs. is the allowable stress, excluding wind.
35100 lbs. is the allowable stress, including wind.

TABLE X.

BEARING ON PINS.

Diam. of Pin.	Bearing.		Diam. of Pin.	Bearing.	
	22000 Lbs. per Sq. In.	28600 Lbs. per Sq. In.		22000 Lbs. per Sq. In.	28600 Lbs. per Sq. In.
2″	44000	57200	7″	154000	200200
2⅛	46800	60800	7⅛	156800	203800
2¼	49500	64400	7¼	159500	207400
2⅜	52300	67900	7⅜	162300	210900
2½	55000	71500	7½	165000	214500
2⅝	57800	75100	7⅝	167800	218100
2¾	60500	78700	7¾	170500	221700
2⅞	63300	82200	7⅞	173300	225200
3	66000	85800	8	176000	228800
3⅛	68800	89400	8⅛	178800	232400
3¼	71500	93000	8¼	181500	236000
3⅜	74300	96500	8⅜	184300	239500
3½	77000	100100	8½	187000	243100
3⅝	79800	103700	8⅝	189800	246700
3¾	82500	107300	8¾	192500	250300
3⅞	85300	110800	8⅞	195300	253800
4	88000	114400	9	198000	257400
4⅛	90800	118000	9⅛	200800	261000
4¼	93500	121600	9¼	203500	264600
4⅜	96300	125100	9⅜	206300	268100
4½	99000	128700	9½	209000	271700
4⅝	101800	132300	9⅝	211800	275300
4¾	104500	135900	9¾	214500	278900
4⅞	107300	139400	9⅞	217300	282400
5	110000	143000	10	220000	286000
5⅛	112800	146600	10⅛	222800	289600
5¼	115500	150200	10¼	225500	293200
5⅜	118300	153700	10⅜	228300	296700
5½	121000	157300	10½	231000	300300
5⅝	123800	160900	10⅝	233800	303900
5¾	126500	164500	10¾	236500	307500
5⅞	129300	168000	10⅞	239300	311000
6	132000	171600	11	242000	314600
6⅛	134800	175200	11⅛	244800	318200
6¼	137500	178800	11¼	247500	321800
6⅜	140300	182300	11⅜	250300	325300
6½	143000	185900	11½	253000	328900
6⅝	145800	189500	11⅝	255800	332500
6¾	148500	193100	11¾	258500	336100
6⅞	151300	196600	11⅞	261300	339600

NOTE.—22000 lbs. per sq. in. is the allowable stress excluding wind.
28600 " " " " " " " including "

TABLE XI.

INTENSITIES FOR FORKED ENDS AND EXTENSION-PLATES OF COMPRESSION-MEMBERS.

Formula: $P = 10000 - 300\frac{l}{t}$.

$\frac{l}{t}$	P	$\frac{l}{t}$	P	$\frac{l}{t}$	P
1	9700	11	6700	21	3700
2	9400	12	6400	22	3400
3	9100	13	6100	23	3100
4	8800	14	5800	24	2800
5	8500	15	5500	25	2500
6	8200	16	5200	26	2200
7	7900	17	4900	27	1900
8	7600	18	4600	28	1600
9	7300	19	4300	29	1300
10	7000	20	4000	30	1000

TABLE XII.

SHEARING AND BEARING VALUES OF RIVETS.

Diameter of Rivet.	Single Shear. 12000 S 9000 F	Bearing Value.	Shop or Field.	Bearing Value for Different Thicknesses of Plates.											
				$\frac{1}{4}''$	$\frac{5}{16}''$	$\frac{3}{8}''$	$\frac{7}{16}''$	$\frac{1}{2}''$	$\frac{9}{16}''$	$\frac{5}{8}''$	$\frac{11}{16}''$	$\frac{3}{4}''$	$\frac{13}{16}''$	$\frac{7}{8}''$	$\frac{15}{16}''$
$\frac{1}{2}''$	S : 2356	22000	S	2750	3437	4125	4812								
	F : 1884		F	2200	2750	3300	3850								
$\frac{5}{8}$	S : 3682	"	S	3437	4296	5155	6014	6875	7734						
	F : 2945		F	2750	3437	4124	4801	5500	6187						
$\frac{3}{4}$	S : 5300	"	S	4125	5156	6187	7218	8250	9281	10312	11343				
	F : 4241		F	3300	4125	4950	5775	6600	7425	8250	9075				
$\frac{7}{8}$	S : 7216	"	S	4812	6015	7218	8421	9625	10828	12031	13234	14437	15640		
	F : 5772		F	3850	4812	5775	6737	7700	8663	9625	10597	11550	12512		
$\frac{15}{16}$	S : 8284	"	S	5156	6445	7734	9023	10312	11601	12890	14179	15468	16757	18046	
	F : 6626		F	4125	5156	6187	7219	8250	9281	10312	11343	12375	13406	14437	
1	S : 9425	"	S	5500	6875	8250	9625	11000	12375	13750	15125	16500	17875	19250	20625
	F : 7540		F	4400	5500	6600	7700	8800	9900	11000	12100	13200	14300	15400	16500

S : Shop-driven rivets. F : Field-driven rivets.

TABLE XIII.

COEFFICIENTS OF $W \tan \theta$ FOR BOTH COMPRESSION AND TENSION STRESSES IN BOTTOM CHORDS OF THROUGH-BRIDGES AND TOP CHORDS OF DECK-BRIDGES, DUE TO WIND LOADS APPLIED TO SAID CHORDS, WHEN THE LATERAL SYSTEM IS OF DOUBLE CANCELLATION.

Number of Panels in Span.	Number of Panel from End of Span.												
	1	2	3	4	5	6	7	8	9	10	11	12	13
4	¾	1¾											
5	1	2½	3										
6	1¼	3¼	4¼										
7	1½	4	5½	6									
8	1¾	4¾	6¾	7¾									
9	2	5½	8	9½	10								
10	2¼	6¼	9¼	11¼	12¼								
11	2½	7	10½	13	14½	15							
12	2¾	7¾	11¾	14¾	16¾	17¾							
13	3	8½	13	16½	19	20½	21						
14	3¼	9¼	14¼	18¼	21¼	23¼	24¼						
15	3½	10	15½	20	23½	26	27½	28					
16	3¾	10¾	16¾	21¾	25¾	28¾	30¾	31¾					
17	4	11½	18	23½	28	31½	34	35½	36				
18	4¼	12¼	19¼	25¼	30¼	34¼	37¼	39¼	40¼				
19	4½	13	20½	27	32½	37	40½	43	44½	45			
20	4¾	13¾	21¾	28¾	34¾	39¾	43¾	46¾	48¾	49¾			
21	5	14½	23	30½	37	42½	47	50½	53	54½	55		
22	5¼	15¼	24¼	32¼	39¼	45¼	50¼	54¼	57¼	59¼	60¼		
23	5½	16	25½	34	41½	48	53½	58	61½	64	65½	66	
24	5¾	16¾	26¾	35¾	43¾	50¾	56¾	61¾	65¾	68¾	70¾	71¾	
25	6	17½	28	37½	46	53½	60	65½	70	73½	76	77½	78
26	6¼	18¼	29¼	39¼	48¼	56¼	63¼	69¼	74¼	78¼	81¼	83¼	84¼

TABLE XIV.

COEFFICIENTS OF $\frac{W \sec \theta}{n}$ (WHERE n = NO. OF PANELS IN SPAN) FOR WIND-LOAD STRESSES IN THE DIAGONALS OF LATERAL SYSTEMS OF SINGLE CANCELLATION. THESE COEFFICIENTS APPLY TO LATERAL SYSTEMS COMPOSED OF INTERSECTING DIAGONAL RODS OR OF SINGLE DIAGONAL STRUTS.

No. of Panels in Span.	Number of Panel from End of Span.												
	1	2	3	4	5	6	7	8	9	10	11	12	13
4	6	3											
5	10	6	3										
6	15	10	6										
7	21	15	10	6									
8	28	21	15	10									
9	36	28	21	15	10								
10	45	36	28	21	15								
11	55	45	36	28	21	15							
12	66	55	45	36	28	21							
13	78	66	55	45	36	28	21						
14	91	78	66	55	45	36	28						
15	105	91	78	66	55	45	36	28					
16	120	105	91	78	66	55	45	36					
17	136	120	105	91	78	66	55	45	36				
18	153	136	120	105	91	78	66	55	45				
19	171	153	136	120	105	91	78	66	55	45			
20	190	171	153	136	120	105	91	78	66	55			
21	210	190	171	153	136	120	105	91	78	66	55		
22	231	210	190	171	153	136	120	105	91	78	66		
23	253	231	210	190	171	153	136	120	105	91	78	66	
24	276	253	231	210	190	171	153	136	120	105	91	78	
25	300	276	253	231	210	190	171	153	136	120	105	91	78
26	325	300	276	253	231	210	190	171	153	136	120	105	91

NOTE.—For the stresses in diagonals of lateral systems of double cancellation, *i.e.*, those systems in which the diagonals are composed of intersecting struts, divide the coefficients in the above table by two.

TABLE XV

COEFFICIENTS OF W TAN θ FOR COMPRESSION-STRESSES IN WINDWARD BOTTOM CHORDS OF THROUGH-BRIDGES, AND WINDWARD TOP CHORDS OF DECK-BRIDGES, DUE TO WIND LOADS APPLIED DIRECTLY TO SAID CHORDS, WHEN THE LATERAL SYSTEM IS OF SINGLE CANCELLATION. THE TENSILE STRESSES IN LEEWARD CHORDS ARE NUMERICALLY EQUAL TO THE COMPRESSION STRESSES GIVEN IN THE TABLE FOR ONE PANEL NEARER END OF SPAN.

No. of Panels in Span.	Number of Panel from End of Span.												
	1	2	3	4	5	6	7	8	9	10	11	12	13
4	1½	2											
5	2	3	3										
6	2½	4	4½										
7	3	5	6	6									
8	3½	6	7½	8									
9	4	7	9	10	10								
10	4½	8	10½	12	12½								
11	5	9	12	14	15	15							
12	5½	10	13½	16	17½	18							
13	6	11	15	18	20	21	21						
14	6½	12	16½	20	22½	24	24½						
15	7	13	18	22	25	27	28	28					
16	7½	14	19½	24	27½	30	31½	32					
17	8	15	21	26	30	33	35	36	36				
18	8½	16	22½	28	32½	36	38½	40	40½				
19	9	17	24	30	35	39	42	44	45	45			
20	9½	18	25½	32	37½	42	45½	48	49½	50			
21	10	19	27	34	40	45	49	52	54	55	55		
22	10½	20	28½	36	42½	48	52½	56	58½	60	60½		
23	11	21	30	38	45	51	56	60	63	65	66	66	
24	11½	22	31½	40	47½	54	59½	64	67½	70	71½	72	
25	12	23	33	42	50	57	63	68	72	75	77	78	78
26	12½	24	34½	44	52½	60	66½	72	76½	80	82½	84	84½

TABLE XVI.

INTENSITIES OF WORKING-STRESSES FOR VARIOUS MATERIALS.

TENSION-STRESSES.

Eye-bars	18000 lbs. per sq. in.
Shapes	16000 " " " "
Flanges of floor-beams and stringers (counting in ⅛ of web)	14000 " " " "
Hip verticals (eye-bars)	16000 " " " "
" " (shapes) and hanger-plates *	14000 " " " "
Adjustable members, soft steel	16000 " " " "
" " wrought iron	13000 " " " "
Lateral rods	18000 " " " "
" shapes	16000 " " " "

COMPRESSION-STRESSES.

Top-chords	18000 lbs. $- 70\frac{l}{r}$ per sq. in.
Inclined end posts	18000 lbs. $- 80\frac{l}{r}$ " " "
Intermediate posts and subdiagonals	16000 lbs. $- 80\frac{l}{r}$ " " "
Lateral struts (no impact for wind loads)	16000 lbs. $- 60\frac{l}{r}$ " " "
Columns of viaducts (fixed ends)	16000 lbs. $- 60\frac{l}{r}$ " " "

(l = unsupported length; r = radius of gyration, both in same unit.)

Forked ends and extension-plates	10000 lbs. $- 300\frac{l}{t}$ " " "

(l = length in inches from centre of pinhole to first rivet beyond point where full section of member begins; t = thickness of plate.)

Rollers, allowing for impact, static load	$600d$ per lin. in.
" " " moving load	$200d$ " " "

(d = diameter of rollers in inches.)

SHEARING-STRESSES.

Webs of plate-girders, medium steel, net section	10000 lbs. per sq. in.
Pins and rivets	12000 " " " "

BENDING-STRESSES.

Extreme fibre of rolled sections of medium steel, impact included	16000 lbs. per sq. in.
Extreme fibre of timber beams, impact included	2000 " " " "

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* Increase net section through eye 50 per cent over that of body of member.

TABLE XVI—(*Continued.*)

INTENSITIES OF WORKING-STRESSES FOR VARIOUS MATERIALS

$$\text{Impact, railway bridges,}\quad I = \frac{400}{L + 500}.$$

$$\text{Impact, highway bridges,}\quad I = \frac{100}{L + 150}.$$

(L = Length in feet of span.)

For reversing-stresses figure the areas required for both tension and compression and add ¾ of the lesser area to the greater.

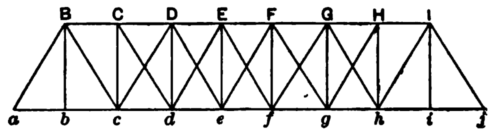
For combined dead, live, and wind load stresses strain 30 per cent higher than for dead and live load only.

The effect of reversion of stresses in case of wind loads is to be ignored.

No impact is to be added for centrifugal and traction loads.

TABLE XVII.

MAXIMUM STRESSES UNDER DEAD AND LIVE LOADS IN PRATT TRUSSES.



W = Dead Load per panel.
L = Live Load per panel.

For any other truss, letter vertices in manner shown.
The Live and Dead Loads are uniform per foot of span.
Panels of through and deck bridges are of equal length.
The Dead Load is assumed as concentrated at lower vertices of trusses for through-bridges and at upper vertices of trusses for deck-bridges.

Member.	12-panel Truss.	11-panel Truss.	10-panel Truss.	9-panel Truss.	8-panel Truss.	Multiply by
aB	$W\ 5.5 + L\ 5.5$	$W\ 5 + L\ 5$	$W\ 4.5 + L\ 4.5$	$W\ 4 + L\ 4$	$W\ 3.5 + L\ 3.5$	Length of member divided by depth of truss.
Bc	" $4.5 +$ " $\frac{55}{12}$	" $4 +$ " $\frac{45}{11}$	" $3.5 +$ " 3.6	" $3 +$ " $\frac{28}{9}$	" $2.5 +$ " $\frac{21}{8}$	
Cd	" $3.5 +$ " $\frac{45}{12}$	" $3 +$ " $\frac{36}{11}$	" $2.5 +$ " 2.8	" $2 +$ " $\frac{21}{9}$	" $1.5 +$ " $\frac{15}{8}$	
De	" $2.5 +$ " $\frac{36}{12}$	" $2 +$ " $\frac{28}{11}$	" $1.5 +$ " 2.1	" $1 +$ " $\frac{15}{9}$	" $0.5 +$ " $\frac{10}{8}$	
Ef	" $1.5 +$ " $\frac{28}{12}$	" $1 +$ " $\frac{21}{11}$	" $0.5 +$ " 1.5	" $0 +$ " $\frac{10}{9}$	$-$" $0.5 +$ " $\frac{6}{8}$	
Fg	" $0.5 +$ " $\frac{21}{12}$	" $0 +$ " $\frac{15}{11}$	$-$" $0.5 +$ " 1.0	$-$ " $1 +$ " $\frac{6}{9}$	$-$" $1.5 +$ " $\frac{3}{8}$	
Gh	$-$" $0.5 +$ " $\frac{15}{12}$	$-$ " $1 +$ " $\frac{10}{11}$	$-$" $1.5 +$ " 0.6	$-$ " $2 +$ " $\frac{3}{9}$		
Hi	$-$" $1.5 +$ " $\frac{10}{12}$	$-$ " $2 +$ " $\frac{6}{11}$				

MAXIMUM STRESSES UNDER DEAD AND LIVE LOADS IN PRATT TRUSSES—*Continued.*

Member.		12-panel Truss.	11-panel Truss.	10-panel Truss.	9-panel Truss.	8-panel Truss.	Multiply by
	abc	W 5.5 + L 5.5	W 5 + L 5	W 4.5 + L 4.5	W 4 + L 4	W 3.5 + L 3.5	Panel length divided by depth of truss.
BC	*cd*	" 10.0 + " 10.0	" 9 + " 9	" 8.0 + " 8.0	" 7 + " 7	" 6.0 + " 6.0	
CD	*de*	" 13.5 + " 13.5	" 12 + " 12	" 10.5 + " 10.5	" 9 + " 9	" 7.5 + " 7.5	
DE	*ef*	" 16.0 + " 16.0	" 14 + " 14	" 12.0 + " 12.0	" 10 + " 10	" 8.0 + " 8.0	
EF	*fg*	" 17.5 + " 17.5	" 15 + " 15	" 12.5 + " 12.5			
FG		" 18.0 + " 18.0					
Through	Deck						
	Cc	" 4.5 + " $\frac{55}{12}$	" 4 + " $\frac{45}{11}$	" 3.5 + " 3.6	" 3 + " $\frac{28}{9}$	" 2.5 + " $\frac{21}{8}$	Unity.
Cc	*Dd*	" 3.5 + " $\frac{45}{12}$	" 3 + " $\frac{36}{11}$	" 2.5 + " 2.8	" 2 + " $\frac{21}{9}$	" 1.5 + " $\frac{15}{8}$	
Dd	*Ee*	" 2.5 + " $\frac{36}{12}$	" 2 + " $\frac{28}{11}$	" 1.5 + " 2.1	" 1 + " $\frac{15}{9}$	" 0.5 + " $\frac{10}{8}$	
Ee	*Ff*	" 1.5 + " $\frac{28}{12}$	" 1 + " $\frac{21}{11}$	" 0.5 + " 1.5	" 0 + " $\frac{10}{9}$	−" 0.5 + " $\frac{6}{8}$	
Ff	*Gg*	" 0.5 + " $\frac{21}{12}$	" 0 + " $\frac{15}{11}$	−" 0.5 + " 1.0			
Gg		−" 0.5 + " $\frac{15}{12}$					

Member.		7-panel Truss.	6-panel Truss.	5-panel Truss.	4-panel Truss.	3-panel Truss.	Multiply by
	aB	W 3 + L 3	W 2.5 + L 2.5	W 2 + L 2.0	W 1.5 + L 1.5	W 1 + L 1	Length of member divided by depth of truss.
	Bc	" 2 + " $\frac{15}{7}$	" 1.5 + " $\frac{10}{6}$	" 1 + " 1.2	" 0.5 + " $\frac{3}{4}$	" 0 + " $\frac{1}{3}$	
	Cd	" 1 + " $\frac{10}{7}$	" 0.5 + " 1.0	" 0 + " 0 6	−" 0.5 + " $\frac{1}{4}$		
	De	" 0 + " $\frac{6}{7}$	−" 0.5 + " 0.5	− " 1 + " 0.2			
	Ef	− " 1 + " $\frac{3}{7}$					
	abc	" 3 + " 3	" 2.5 + " 2.5	" 2 + " 2	" 1.5 + " 1.5	" 1 + " 1	Panel length div. by depth of truss.
BC	*cd*	" 5 + " 5	" 4.0 + " 4.0	" 3 + " 3	" 2.0 + " 2.0	" 1 + " 1	
CD	*de*	" 6 + " 6	" 4.5 + " 4.5				
Through	Deck						
	Cc	" 2 + " $\frac{15}{7}$	" 1.5 + " $\frac{10}{6}$	" 1 + " 1.2	" 0.5 + " $\frac{3}{4}$		Unity.
Cc	*Dd*	" 1 + " $\frac{10}{7}$	" 0.5 + " 1.0	" 0 + " 0.6	−" 0.5 + " $\frac{1}{4}$		
Dd	*Ee*	" 0 + " $\frac{6}{7}$	−" 0.5 + " 0.5				

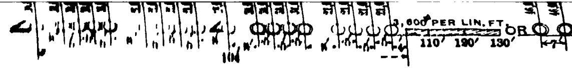
Table XVIII.

SUPERELEVATIONS OF OUTER RAIL ON CURVES.

$$E = \frac{.3277\,V^2}{R}.$$

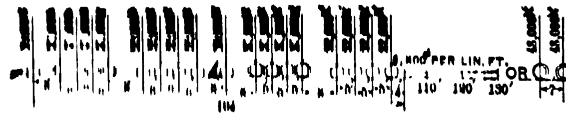
E = Elevation in feet.
V = Velocity in miles per hour.
R = Radius in feet.

Degree.	10	15	20	25	30	35	40	45	50	55	60	Degree.
	Speed in Miles per Hour.											
1	1/16″	1/8″	1/4″	7/16″	5/8″	13/16″	1 1/16″	1 [illegible]″	1 1/16″	2 1/16″	2 [illegible]″	1
2	1/8	5/16	9/16	7/8	1 [illegible]	1 [illegible]	2 1/16	2 [illegible]	3 [illegible]	4 [illegible]	4 [illegible]	2
3	3/16	7/16	13/16	1 5/16	1 [illegible]	2 [illegible]	3 [illegible]	4 [illegible]	5 [illegible]	6 [illegible]	7 [illegible]	3
4	1/4	[illegible]	1 1/16	1 [illegible]	2 [illegible]	3 [illegible]	4 [illegible]	5 [illegible]	6 [illegible]	8 [illegible]	9 [illegible]	4
5	[illegible]	[illegible]	1 [illegible]	2 [illegible]	3 [illegible]	4 1/16	5 [illegible]	6 [illegible]	8 [illegible]	10 [illegible]	12 1/16	5
6	7/16	[illegible]	1 [illegible]	2 9/16	3 [illegible]	5.0	6 [illegible]	8 [illegible]	10 [illegible]	13 [illegible]		
7	1/2	1 1/16	1 [illegible]	3.0	4 [illegible]	5 [illegible]	7 [illegible]	9 [illegible]	11 [illegible]			
8	9/16	1 [illegible]	2 3/16	3 [illegible]	4 [illegible]	6 [illegible]	8 [illegible]	10 [illegible]				
9	[illegible]	1 [illegible]	2 [illegible]	3 [illegible]	5 [illegible]	7 [illegible]	9 [illegible]	12 1/16				
10	[illegible]	1 9/16	2 [illegible]	4 [illegible]	6 [illegible]	8 [illegible]	10 [illegible]					
11	[illegible]	1 [illegible]	3.0	4 [illegible]	6 [illegible]	9 [illegible]	11 [illegible]					
12	[illegible]	1 [illegible]	3 [illegible]	5 [illegible]	7 [illegible]	9 [illegible]	12 [illegible]					
14	[illegible]	2 3/16	3 [illegible]	5 [illegible]	8 [illegible]	11 [illegible]						
16	1 1/16	2 [illegible]	4 [illegible]	6 [illegible]	9 [illegible]	13 1/16						
18	1 [illegible]	2 [illegible]	4 [illegible]	7 [illegible]	10 [illegible]							
20	1 [illegible]	3 [illegible]	5 7/16	8 [illegible]	12.0							
24	1 [illegible]	3 [illegible]	6 [illegible]	10 1/16								
30	2.0	4 9/16	8 1/16	12 7/16								
40	2 11/16	6.0	10 9/16									



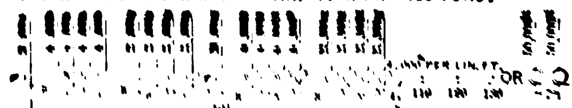
CLASS V.

TOTAL WEIGHT OF ONE ENGINE AND TENDER – 127.5 TONS.



CLASS U.

TOTAL WEIGHT OF ONE ENGINE AND TENDER – 136 TONS.



CLASS T.

TOTAL WEIGHT OF ONE ENGINE AND TENDER – 144.5 TONS.

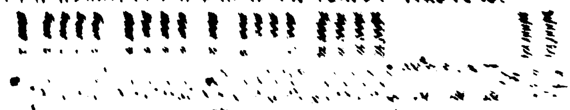


PLATE I

AXLE CONCENTRATIONS

FOR

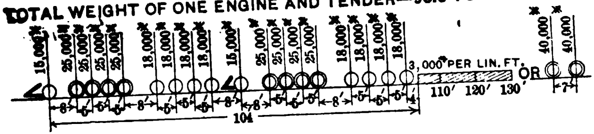
THE COMPROMISE STANDARD SYSTEM

OF

LIVE LOADS FOR RAILWAY BRIDGES.

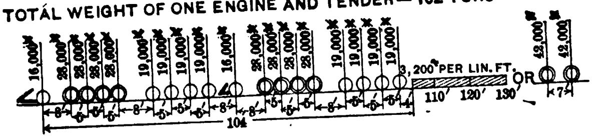
CLASS Z.

TOTAL WEIGHT OF ONE ENGINE AND TENDER=93.5 TONS.



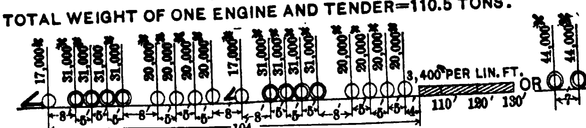
CLASS Y.

TOTAL WEIGHT OF ONE ENGINE AND TENDER=102 TONS.



CLASS X.

TOTAL WEIGHT OF ONE ENGINE AND TENDER=110.5 TONS.



CLASS W

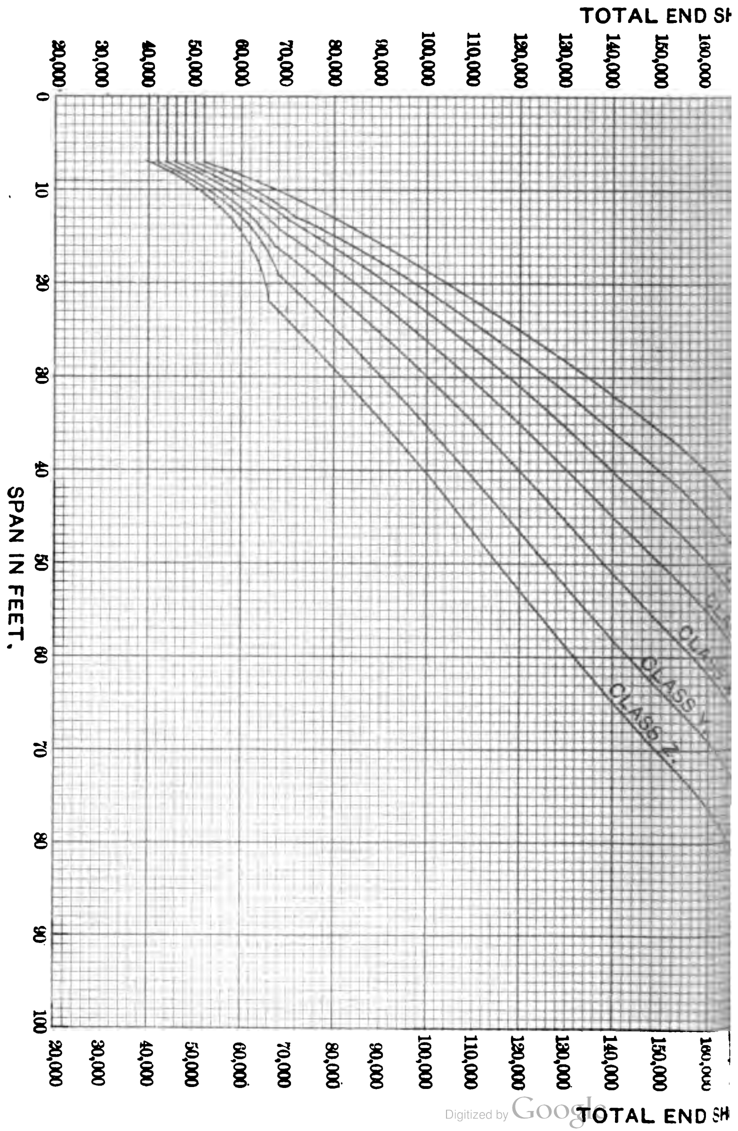

TOTAL END SH
20,000
30,000
40,000
50,000
60,000
70,000
80,000
90,000
100,000
110,000
120,000
130,000
140,000
150,000
160,000
0
10
20
30
40
50
60
70
80
90
100
SPAN IN FEET.
CLASS Y
CLASS Z.
TOTAL END SH

PLATE II.

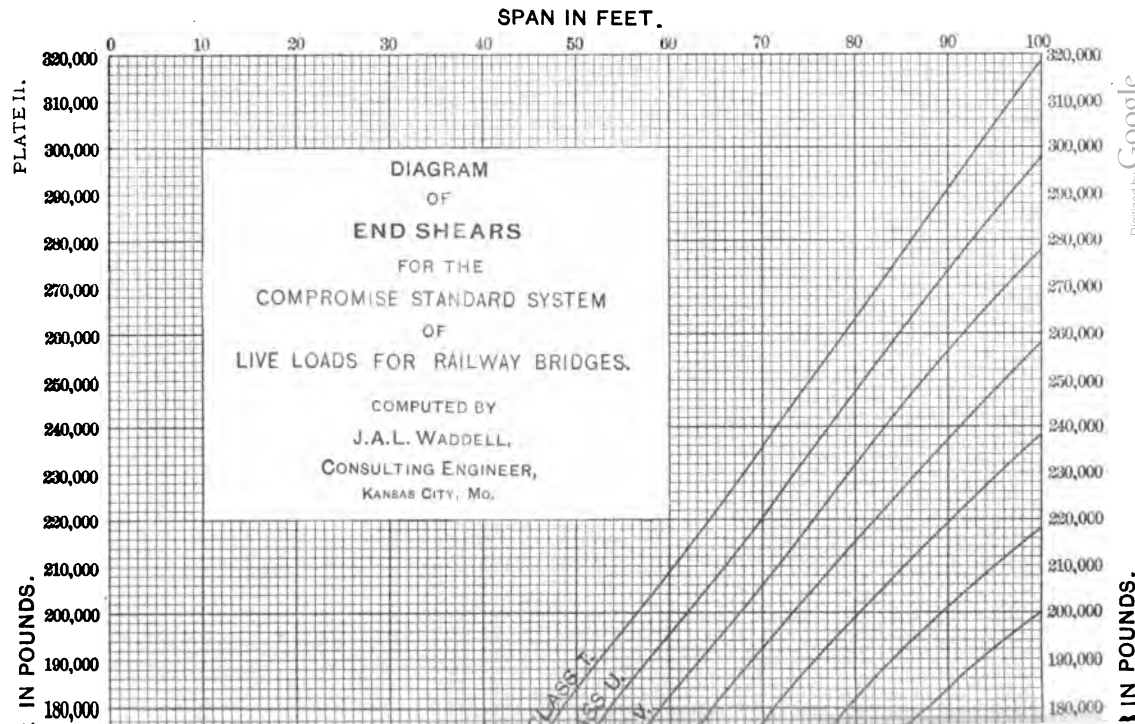
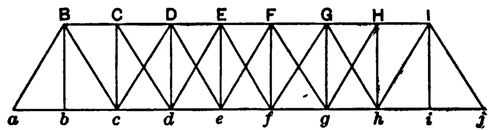

SPAN IN FEET.
0
10
20
30
40
50
60
70
80
90
100
320,000
310,000
300,000
290,000
280,000
270,000
260,000
250,000
240,000
230,000
220,000
210,000
200,000
190,000
180,000
R IN POUNDS.
DIAGRAM
OF
END SHEARS
FOR THE
COMPROMISE STANDARD SYSTEM
OF
LIVE LOADS FOR RAILWAY BRIDGES.
COMPUTED BY
J.A.L. WADDELL,
CONSULTING ENGINEER,
KANSAS CITY, MO.
CLASS T.
ASS U.
V.

TABLE XVII.

MAXIMUM STRESSES UNDER DEAD AND LIVE LOADS IN PRATT TRUSSES.



W = Dead Load per panel.
L = Live Load per panel.

For any other truss, letter vertices in manner shown.
The Live and Dead Loads are uniform per foot of span.
Panels of through and deck bridges are of equal length.
The Dead Load is assumed as concentrated at lower vertices of trusses for through-bridges and at upper vertices of trusses for deck-bridges.

Member.	12-panel Truss.	11-panel Truss.	10-panel Truss.	9-panel Truss.	8-panel Truss.	Multiply by
aB	W 5.5 + L 5 5	W 5 + L 5	W 4.5 + L 4.5	W 4 + L 4	W 3.5 + L 3.5	Length of member divided by depth of truss.
Bc	" 4.5 + " $\frac{55}{12}$	" 4 + " $\frac{45}{11}$	" 3 5 + " 3.6	" 3 + " $\frac{28}{9}$	" 2.5 + " $\frac{21}{8}$	
Cd	" 3.5 + " $\frac{45}{12}$	" 3 + " $\frac{36}{11}$	" 2.5 + " 2.8	" 2 + " $\frac{21}{9}$	" 1.5 + " $\frac{15}{8}$	
De	" 2.5 + " $\frac{36}{12}$	" 2 + " $\frac{28}{11}$	" 1.5 + " 2.1	" 1 + " $\frac{15}{9}$	" 0.5 + " $\frac{10}{8}$	
Ef	" 1.5 + " $\frac{28}{12}$	" 1 + " $\frac{21}{11}$	" 0.5 + " 1 5	" 0 + " $\frac{10}{9}$	−" 0.5 + " $\frac{6}{8}$	
Fg	" 0 5 + " $\frac{21}{12}$	" 0 + " $\frac{15}{11}$	−" 0.5 + " 1.0	− " 1 + " $\frac{6}{9}$	−" 1.5 + " $\frac{3}{8}$	
Gh	−" 0.5 + " $\frac{15}{12}$	− " 1 + " $\frac{10}{11}$	−" 1.5 + " 0.6	− " 2 + " $\frac{3}{9}$		
Hi	−" 1.5 + " $\frac{10}{12}$	− " 2 + " $\frac{6}{11}$				

MAXIMUM STRESSES UNDER DEAD AND LIVE LOADS IN PRATT TRUSSES—*Continued.*

Member.		12-panel Truss.	11-panel Truss.	10-panel Truss.	9-panel Truss.	8-panel Truss.	Multiply by
	abc	W 5.5 + L 5.5	W 5 + L 5	W 4.5 + L 4.5	W 4 + L 4	W 3.5 + L 3.5	Panel length divided by depth of truss.
BC	*cd*	" 10.0 + " 10.0	" 9 + " 9	" 8.0 + " 8.0	" 7 + " 7	" 6.0 + " 6.0	
CD	*de*	" 13.5 + " 13.5	" 12 + " 12	" 10.5 + " 10.5	" 9 + " 9	" 7.5 + " 7.5	
DE	*ef*	" 16.0 + " 16.0	" 14 + " 14	" 12.0 + " 12.0	" 10 + " 10	" 8.0 + " 8.0	
EF	*fg*	" 17.5 + " 17.5	" 15 + " 15	" 12.5 + " 12.5			
FG		" 18.0 + " 18.0					
Through	Deck						
	Cc	" 4.5 + " $\frac{55}{12}$	" 4 + " $\frac{45}{11}$	" 3.5 + " 3.6	" 3 + " $\frac{28}{9}$	" 2.5 + " $\frac{21}{8}$	Unity.
Cc	*Dd*	" 3.5 + " $\frac{45}{12}$	" 3 + " $\frac{36}{11}$	" 2.5 + " 2.8	" 2 + " $\frac{21}{9}$	" 1.5 + " $\frac{15}{8}$	
Dd	*Ee*	" 2.5 + " $\frac{36}{12}$	" 2 + " $\frac{28}{11}$	" 1.5 + " 2.1	" 1 + " $\frac{15}{9}$	" 0.5 + " $\frac{10}{8}$	
Ee	*Ff*	" 1.5 + " $\frac{28}{12}$	" 1 + " $\frac{21}{11}$	" 0.5 + " 1.5	" 0 + " $\frac{10}{9}$	−" 0.5 + " $\frac{6}{8}$	
Ff	*Gg*	" 0.5 + " $\frac{21}{12}$	" 0 + " $\frac{15}{11}$	−" 0.5 + " 1.0			
Gg		−" 0.5 + " $\frac{15}{12}$					

Member.		7-panel Truss.	6-panel Truss.	5-panel Truss.	4-panel Truss.	3-panel Truss.	Multiply by
	aB	W 3 + L 3	W 2.5 + L 2.5	W 2 + L 2.0	W 1.5 + L 1.5	W 1 + L 1	Length of member divided by depth of truss.
	Bc	" 2 + " $\frac{15}{7}$	" 1.5 + " $\frac{10}{6}$	" 1 + " 1.2	" 0.5 + " $\frac{3}{4}$	" 0 + " $\frac{1}{3}$	
	Cd	" 1 + " $\frac{10}{7}$	" 0.5 + " 1.0	" 0 + " 0 6	−" 0.5 + " $\frac{1}{4}$		
	De	" 0 + " $\frac{6}{7}$	−" 0.5 + " 0.5	− " 1 + " 0.2			
	Ef	− " 1 + " $\frac{3}{7}$					
	abc	" 3 + " 3	" 2.5 + " 2.5	" 2 + " 2	" 1.5 + " 1.5	" 1 + " 1	Panel length div. by depth of truss.
BC	*cd*	" 5 + " 5	" 4.0 + " 4.0	" 3 + " 3	" 2.0 + " 2.0	" 1 + " 1	
CD	*de*	" 6 + " 6	" 4.5 + " 4.5				
Through	Deck						
	Cc	" 2 + " $\frac{15}{7}$	" 1.5 + " $\frac{10}{6}$	" 1 + " 1.2	" 0.5 + " $\frac{3}{4}$		Unity.
Cc	*Dd*	" 1 + " $\frac{10}{7}$	" 0.5 + " 1.0	" 0 + " 0.6	−" 0.5 + " $\frac{1}{4}$		
Dd	*Ee*	" 0 + " $\frac{6}{7}$	−" 0.5 + " 0.5				

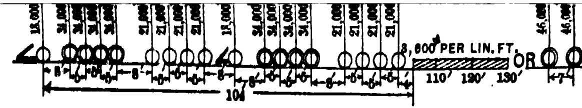
TABLE XVIII.

SUPERELEVATIONS OF OUTER RAIL ON CURVES.

$$E = \frac{.3277\,V^2}{R}.$$

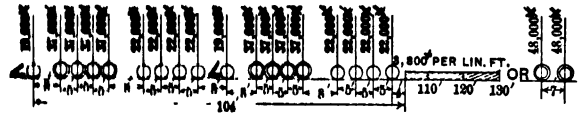
E = Elevation in feet.
V = Velocity in miles per hour.
R = Radius in feet.

Degree.	Speed in Miles per Hour.											Degree.
	10	15	20	25	30	35	40	45	50	55	60	
1	1/16″	1/8″	1/4″	7/16″	5/8″	13/16″	1 1/16″	1 3/8″	1 11/16″	2 1/16″	2 1/2″	1
2	1/8	5/16	9/16	7/8	1 1/4	1 11/16	2 3/16	2 3/4	3 7/16	4 1/8	4 15/16	2
3	3/16	7/16	13/16	1 5/16	1 7/8	2 1/2	3 5/16	4 1/8	5 1/8	6 3/16	7 3/8	3
4	1/4	5/8	1 1/16	1 11/16	2 1/2	3 3/8	4 3/8	5 9/16	6 13/16	8 1/4	9 3/4	4
5	3/8	3/4	1 3/8	2 1/8	3 1/16	4 3/16	5 7/16	6 7/8	8 1/2	10 1/4	12 1/16	5
6	7/16	15/16	1 5/8	2 9/16	3 11/16	5.0	6 9/16	8 1/4	10 1/4	13 3/8		
7	1/2	1 1/16	1 15/16	3.0	4 5/16	5 13/16	7 5/8	9 9/16	11 3/4			
8	9/16	1 1/4	2 3/16	3 3/8	4 7/8	6 9/16	8 11/16	10 7/8				
9	5/8	1 3/8	2 1/2	3 13/16	5 1/2	7 1/2	9 3/4	12 3/16				
10	11/16	1 9/16	2 3/4	4 1/4	6 1/8	8 5/16	10 3/4					
11	3/4	1 11/16	3.0	4 11/16	6 3/4	9 1/8	11 13/16					
12	13/16	1 7/8	3 5/16	5 1/8	7 5/16	9 15/16	12 13/16					
14	15/16	2 3/16	3 13/16	5 15/16	8 9/16	11 1/2						
16	1 1/16	2 1/2	4 3/8	6 13/16	9 11/16	13 1/16						
18	1 1/4	2 3/4	4 15/16	7 5/8	10 7/8							
20	1 3/8	3 1/16	5 7/16	8 1/2	12.0							
24	1 5/8	3 11/16	6 1/2	10 1/16								
30	2.0	4 9/16	8 1/16	12 7/16								
40	2 11/16	6.0	10 9/16									



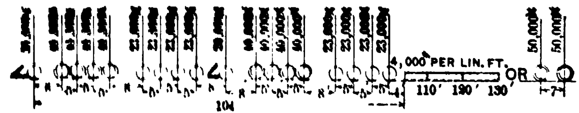
CLASS V.

TOTAL WEIGHT OF ONE ENGINE AND TENDER = 127.5 TONS.



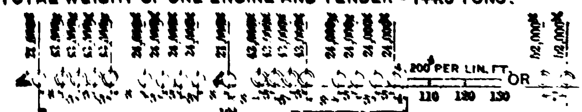
CLASS U.

TOTAL WEIGHT OF ONE ENGINE AND TENDER = 136 TONS.



CLASS T.

TOTAL WEIGHT OF ONE ENGINE AND TENDER = 144.5 TONS.



N.B. These concentrated loads are to be employed only as a standard of reference, and are the loads which were used in obtaining the data for plotting the curves on Plates, Nos. 2, 3 & 4. For finding stresses use the equivalent loads given by said curves.

AXLE CONCENTRATIONS

FOR

THE COMPROMISE STANDARD SYSTEM

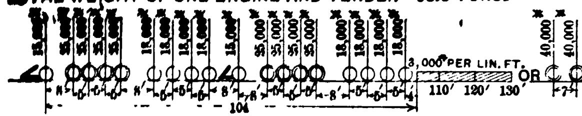
OF

LIVE LOADS FOR RAILWAY BRIDGES.

PLATE I

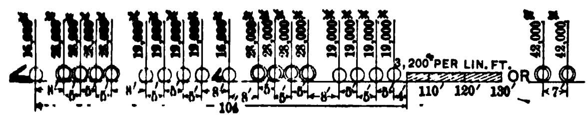
CLASS Z.

TOTAL WEIGHT OF ONE ENGINE AND TENDER=93.5 TONS.



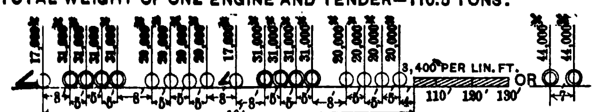
CLASS Y.

TOTAL WEIGHT OF ONE ENGINE AND TENDER= 102 TONS.



CLASS X.

TOTAL WEIGHT OF ONE ENGINE AND TENDER=110.5 TONS.



CLASS W.

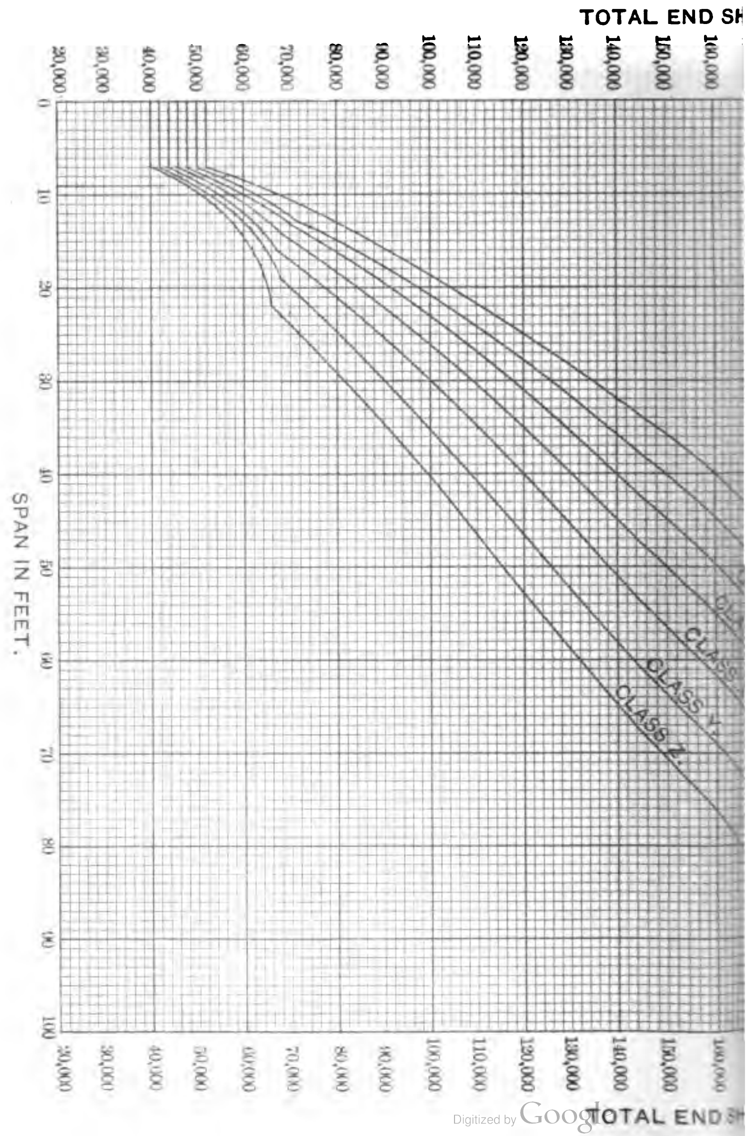
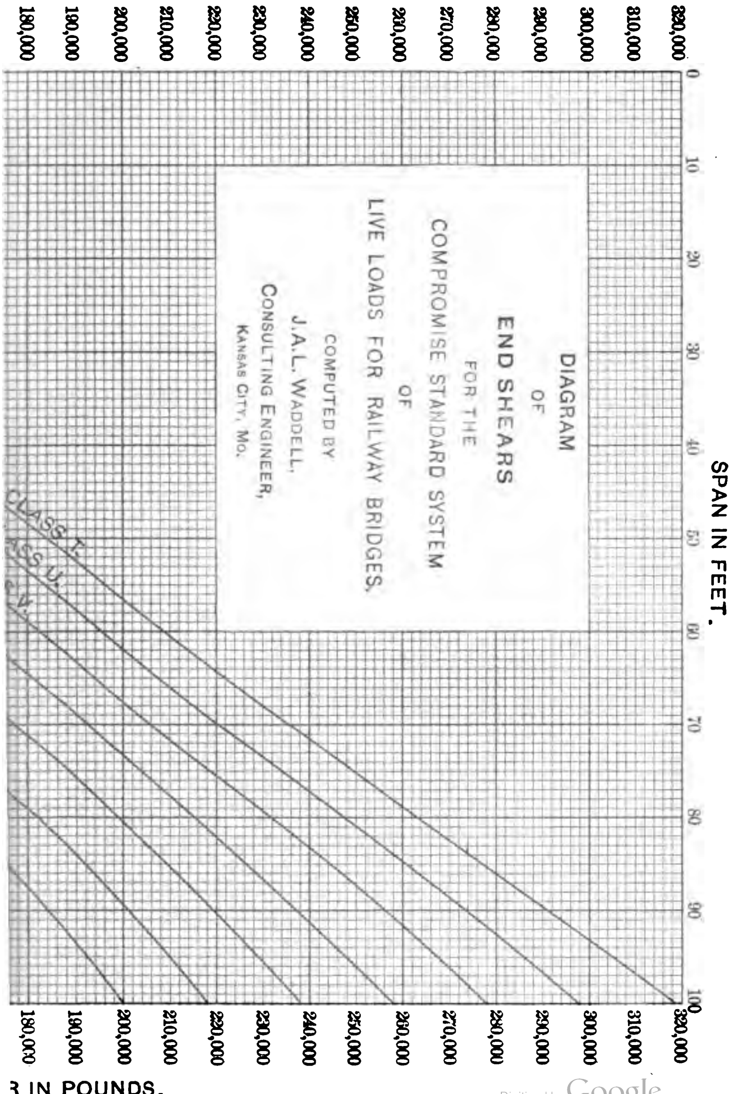
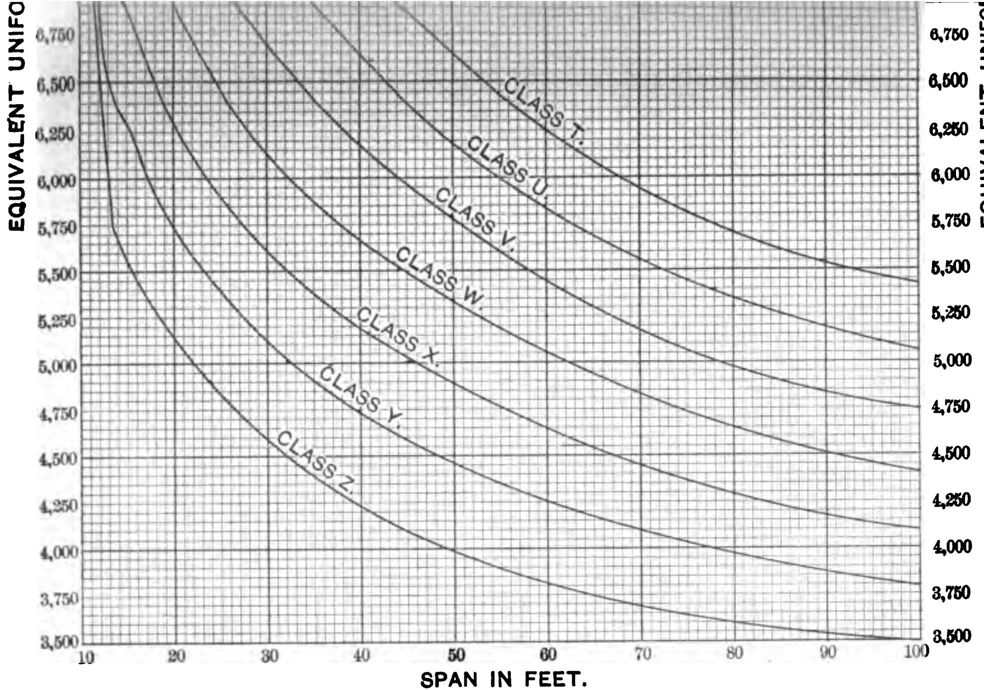

TOTAL END SH
20,000
30,000
40,000
50,000
60,000
70,000
80,000
90,000
100,000
110,000
120,000
130,000
140,000
150,000
160,000
0
10
20
30
40
50
60
70
80
90
100
SPAN IN FEET.
CLASS Y.
CLASS Z.
TOTAL END SH

PLATE II.

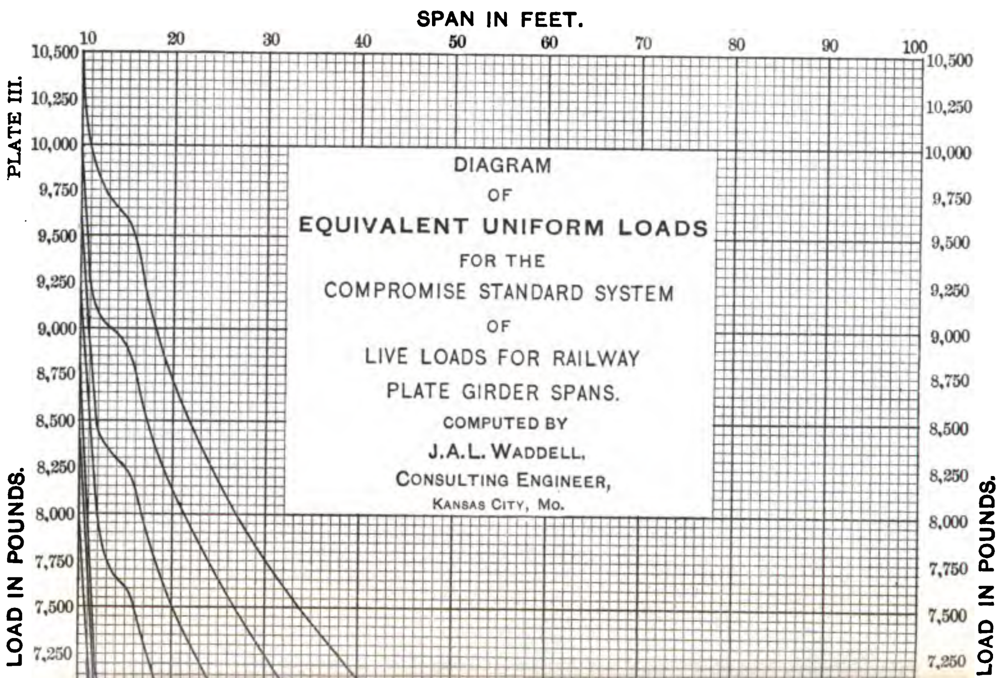


R IN POUNDS.
DIAGRAM
OF
END SHEARS
FOR THE
COMPROMISE STANDARD SYSTEM
OF
LIVE LOADS FOR RAILWAY BRIDGES,
COMPUTED BY
J.A.L. WADDELL,
CONSULTING ENGINEER,
KANSAS CITY, MO.
SPAN IN FEET.
CLASS T.
ASS U.
S V.
0
10
20
30
40
50
60
70
80
90
100
180,000
190,000
200,000
210,000
220,000
230,000
240,000
250,000
260,000
270,000
280,000
290,000
300,000
310,000
320,000
R IN POUNDS.



N.B. For Panels or Spans shorter than 10′, use one of the axle loads of the alternative loading (placing the wheel at mid-length) and the formula M = ¼ Wl.

PLATE III.


SPAN IN FEET.
10
20
30
40
50
60
70
80
90
100
LOAD IN POUNDS.
10,500
10,250
10,000
9,750
9,500
9,250
9,000
8,750
8,500
8,250
8,000
7,750
7,500
7,250
DIAGRAM
OF
EQUIVALENT UNIFORM LOADS
FOR THE
COMPROMISE STANDARD SYSTEM
OF
LIVE LOADS FOR RAILWAY
PLATE GIRDER SPANS.
COMPUTED BY
J.A.L. WADDELL,
CONSULTING ENGINEER,
KANSAS CITY, MO.

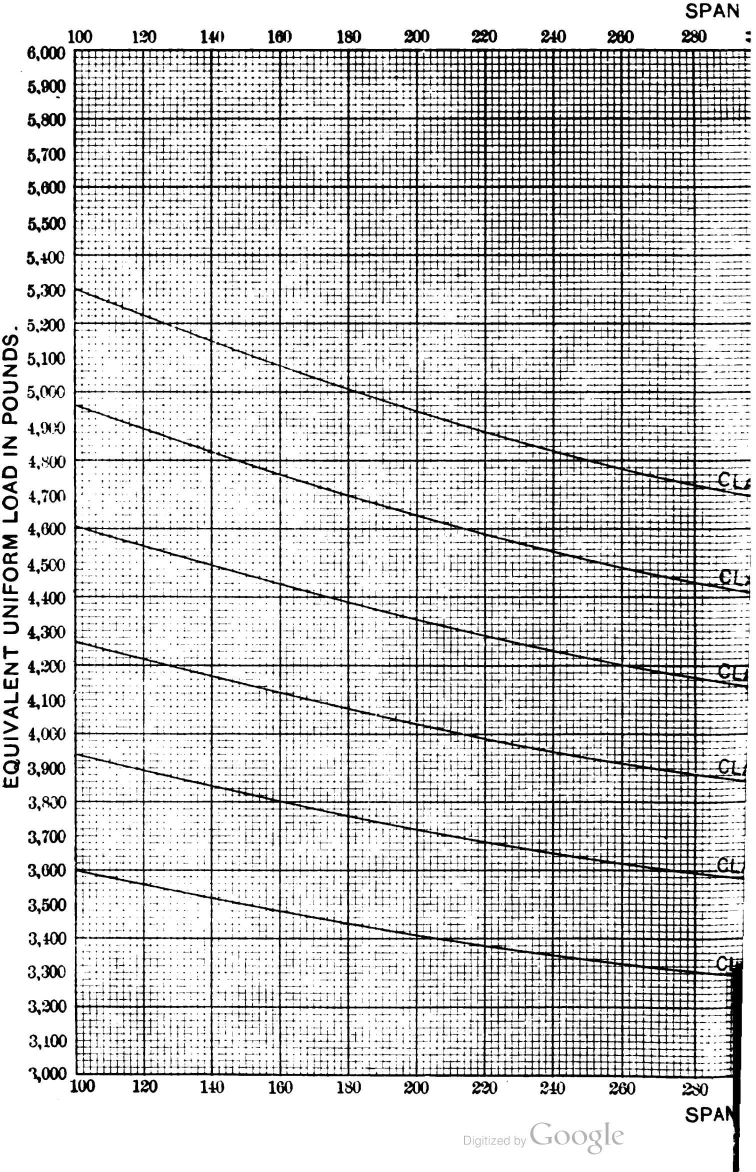
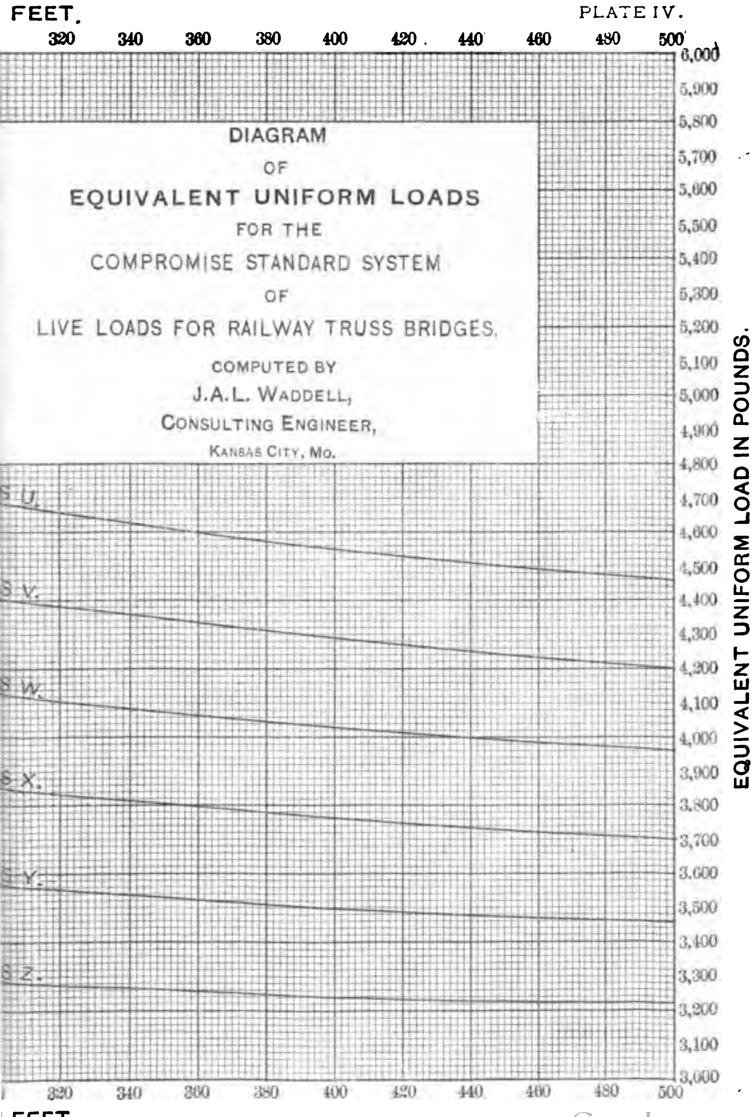
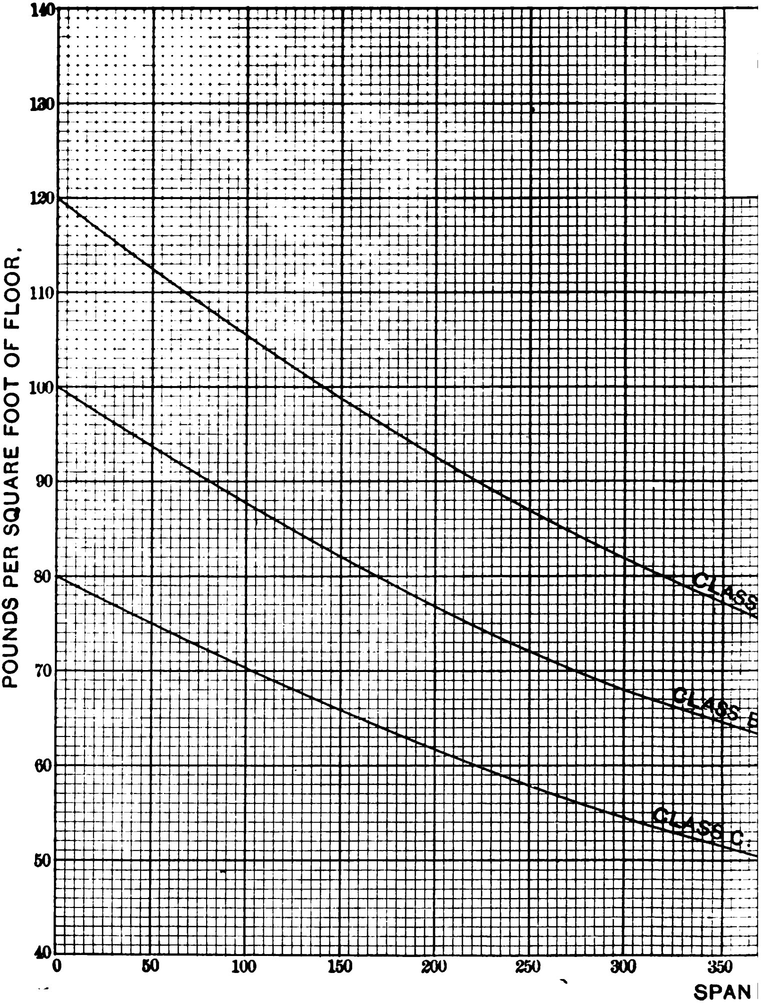

SPAN
100
120
140
160
180
200
220
240
260
280
6,000
5,900
5,800
5,700
5,600
5,500
5,400
5,300
5,200
5,100
5,000
4,900
4,800
4,700
4,600
4,500
4,400
4,300
4,200
4,100
4,000
3,900
3,800
3,700
3,600
3,500
3,400
3,300
3,200
3,100
3,000
EQUIVALENT UNIFORM LOAD IN POUNDS.
SPAN

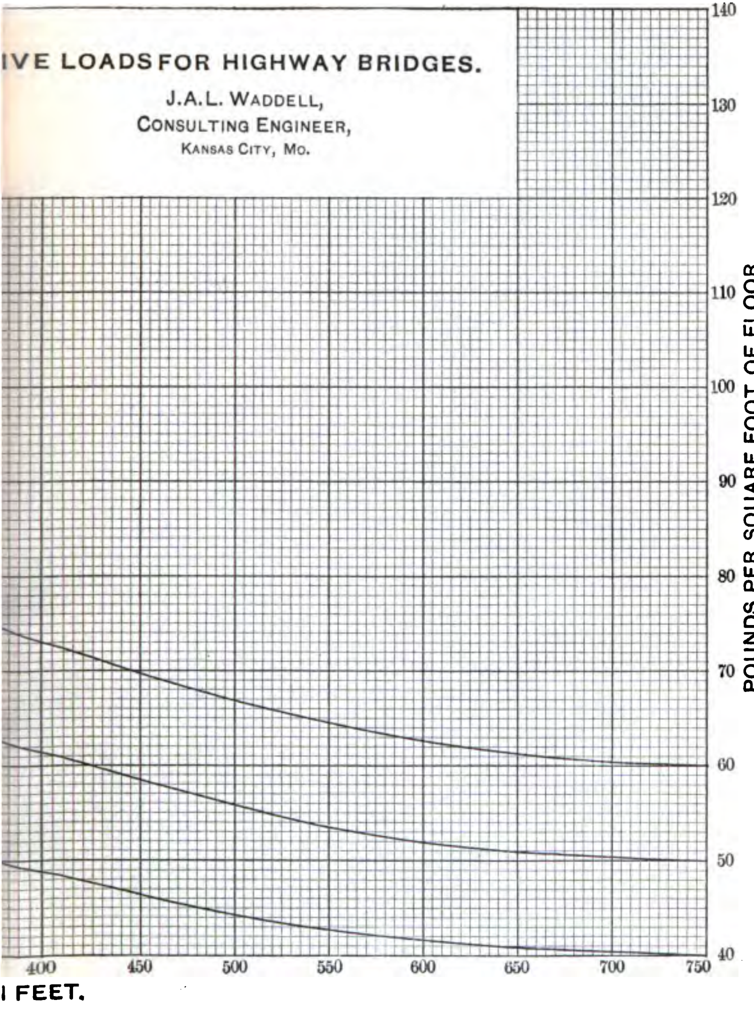
PLATE IV.


FEET.
320
340
360
380
400
420
440
460
480
500
DIAGRAM
OF
EQUIVALENT UNIFORM LOADS
FOR THE
COMPROMISE STANDARD SYSTEM
OF
LIVE LOADS FOR RAILWAY TRUSS BRIDGES.
COMPUTED BY
J.A.L. WADDELL,
CONSULTING ENGINEER,
KANSAS CITY, MO.
S U.
S V.
S W.
S X.
S Y.
S Z.
6,000
5,900
5,800
5,700
5,600
5,500
5,400
5,300
5,200
5,100
5,000
4,900
4,800
4,700
4,600
4,500
4,400
4,300
4,200
4,100
4,000
3,900
3,800
3,700
3,600
3,500
3,400
3,300
3,200
3,100
3,000
EQUIVALENT UNIFORM LOAD IN POUNDS.
FEET.



POUNDS PER SQUARE FOOT OF FLOOR.
140
130
120
110
100
90
80
70
60
50
40
0
50
100
150
200
250
300
350
CLASS
CLASS B
CLASS C.
SPAN

PLATE V.


IVE LOADS FOR HIGHWAY BRIDGES.
J.A.L. WADDELL,
CONSULTING ENGINEER,
KANSAS CITY, MO.
140
130
120
110
100
90
80
70
60
50
40
400
450
500
550
600
650
700
750
POUNDS PER SQUARE FOOT OF FLOOR.
I FEET.

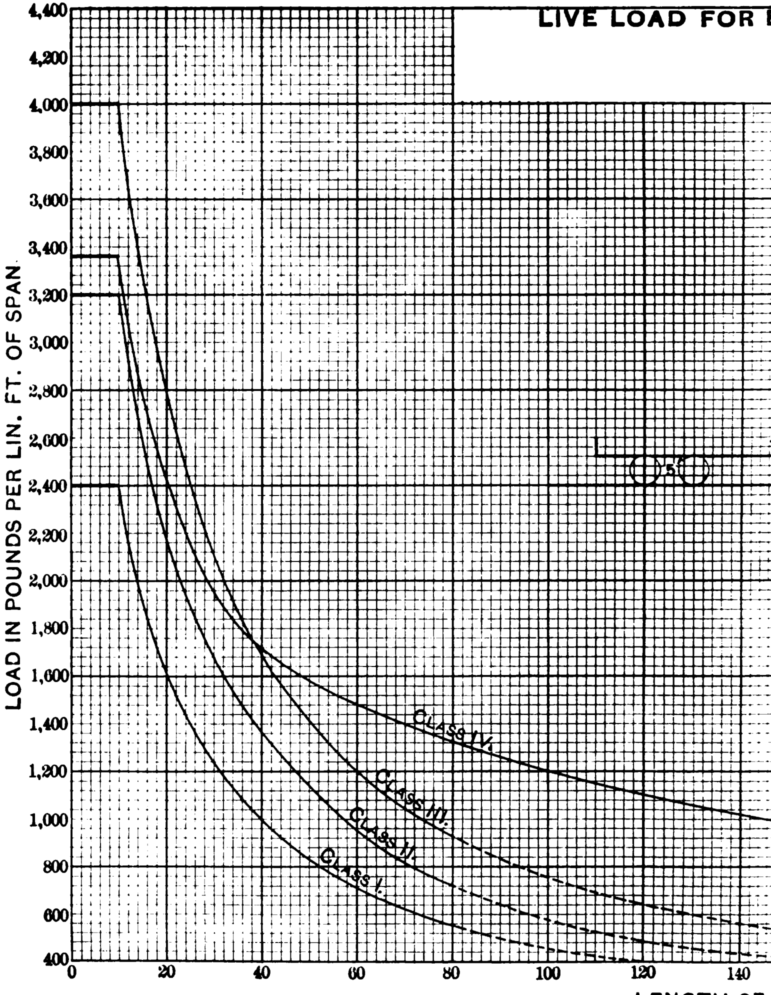
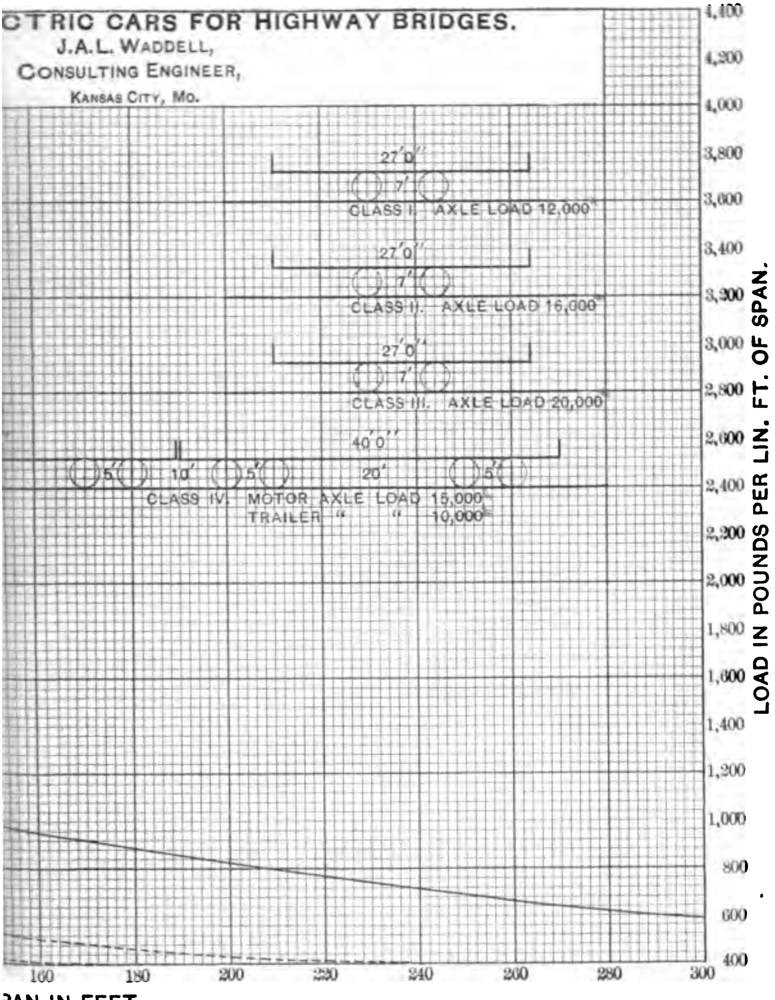

LIVE LOAD FOR E
LOAD IN POUNDS PER LIN. FT. OF SPAN.
4,400
4,200
4,000
3,800
3,600
3,400
3,200
3,000
2,800
2,600
2,400
2,200
2,000
1,800
1,600
1,400
1,200
1,000
800
600
400
0
20
40
60
80
100
120
140
LENGTH OF S
CLASS IV.
CLASS III.
CLASS II.
CLASS I.
5
4

PLATE VI.



CTRIC CARS FOR HIGHWAY BRIDGES.
J.A.L. WADDELL,
CONSULTING ENGINEER,
KANSAS CITY, MO.
27'0"
7'
CLASS I. AXLE LOAD 12,000
27'0"
7'
CLASS II. AXLE LOAD 16,000
27'0"
7'
CLASS III. AXLE LOAD 20,000
40'0"
5' 10' 5' 20' 5'
CLASS IV. MOTOR AXLE LOAD 15,000
TRAILER " " 10,000
4,400
4,200
4,000
3,800
3,600
3,400
3,200
3,000
2,800
2,600
2,400
2,200
2,000
1,800
1,600
1,400
1,200
1,000
800
600
400
LOAD IN POUNDS PER LIN. FT. OF SPAN.
160
180
200
220
240
260
280
300
PAN IN FEET.

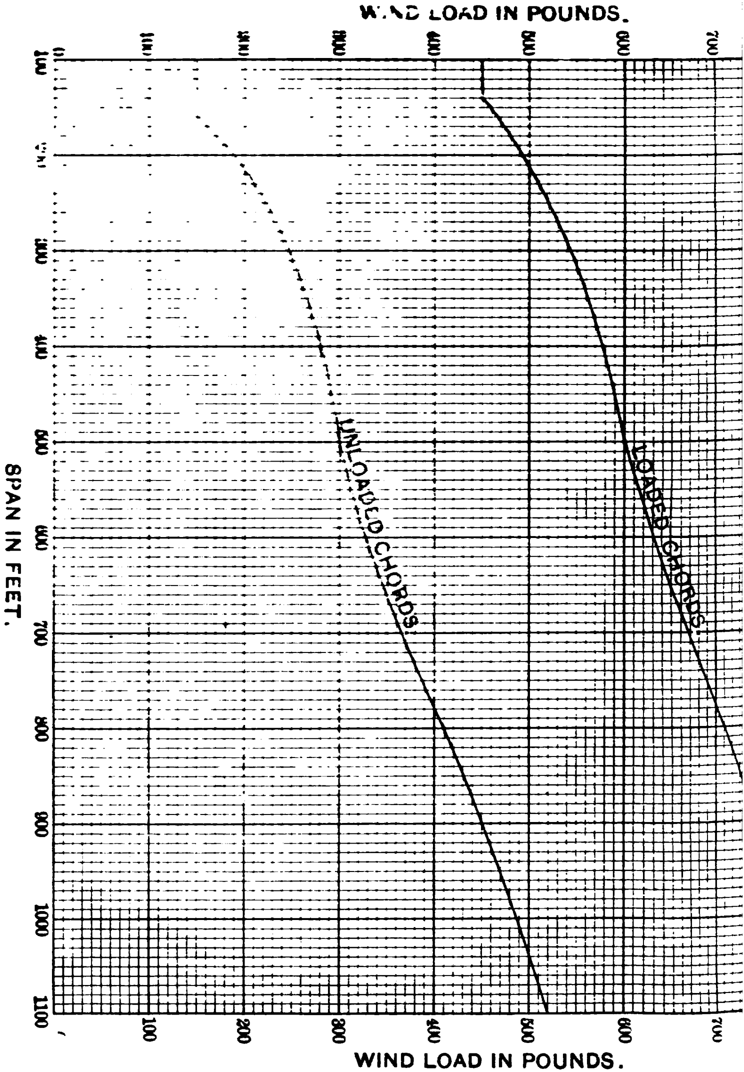

WIND LOAD IN POUNDS.
SPAN IN FEET.
100
200
300
400
500
600
700
800
900
1000
1100
0
100
200
300
400
500
600
700
UNLOADED CHORDS.
LOADED CHORDS.
WIND LOAD IN POUNDS.

PLATE VII.

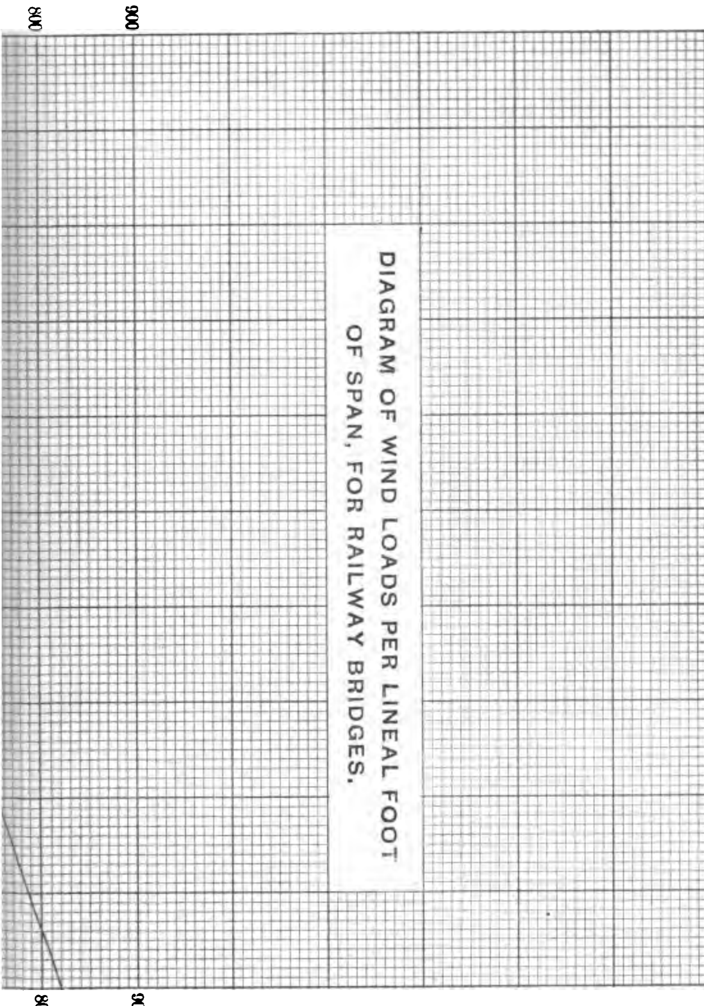
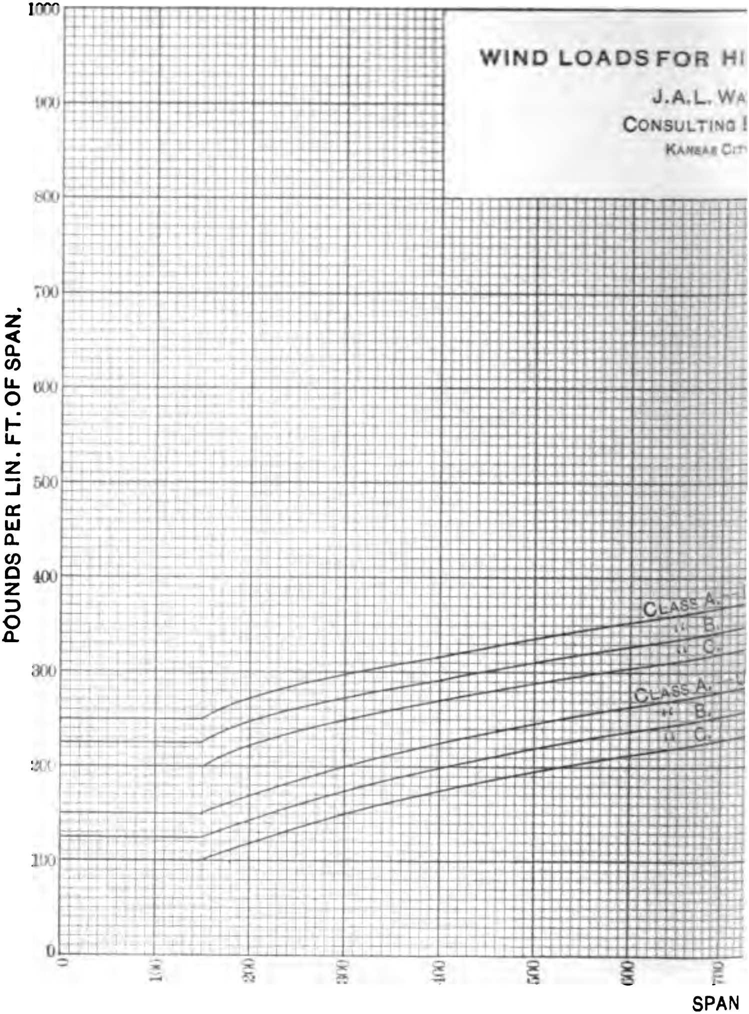
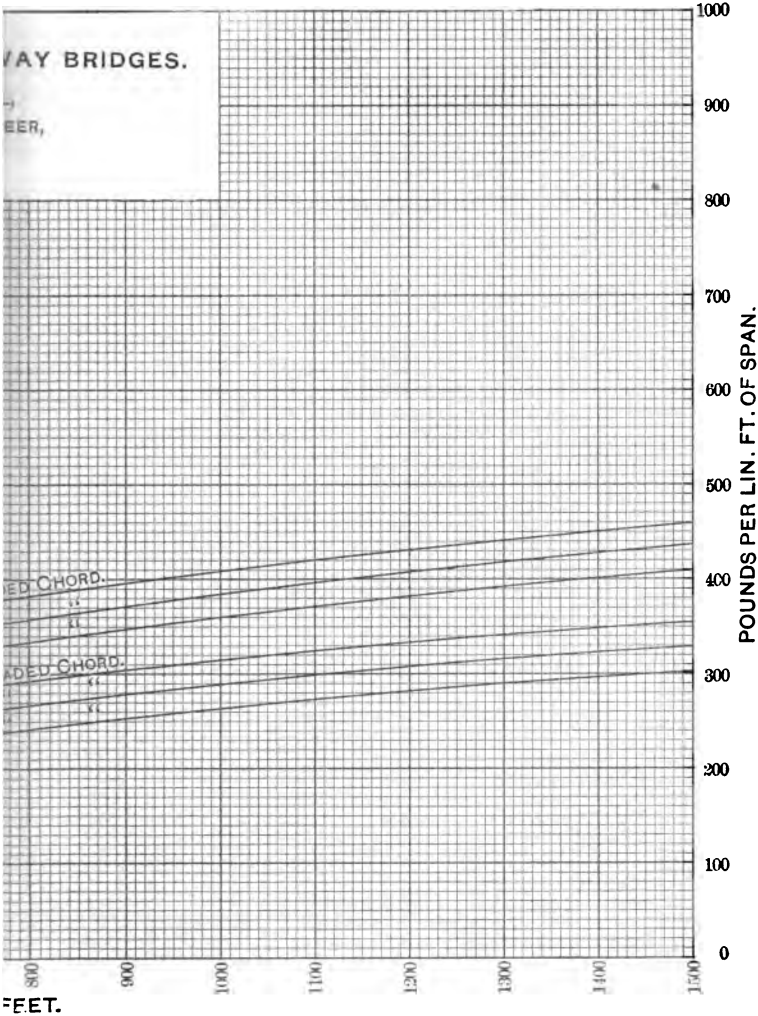


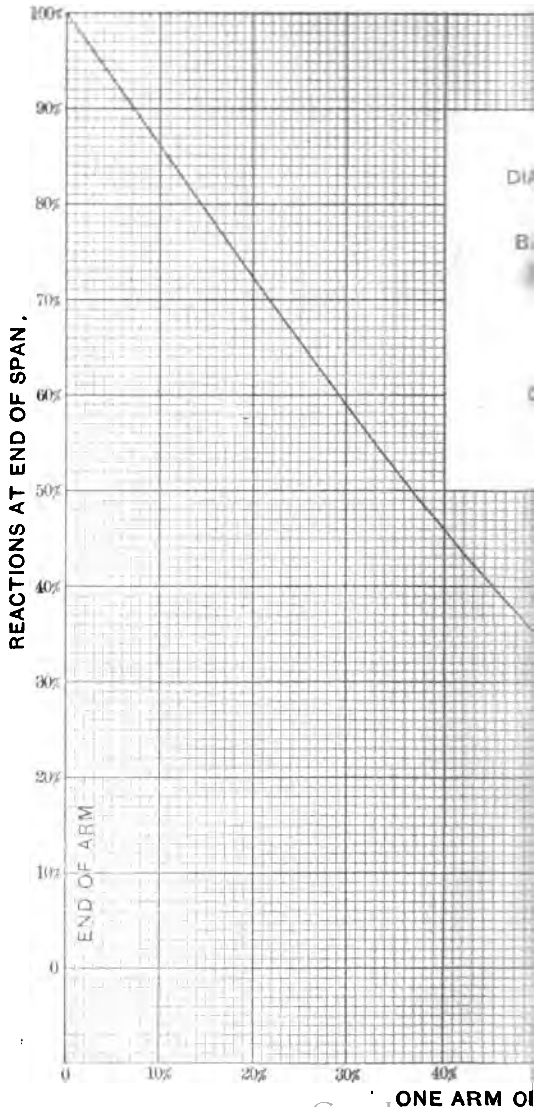
DIAGRAM OF WIND LOADS PER LINEAL FOOT
OF SPAN, FOR RAILWAY BRIDGES.
800
900
800
900

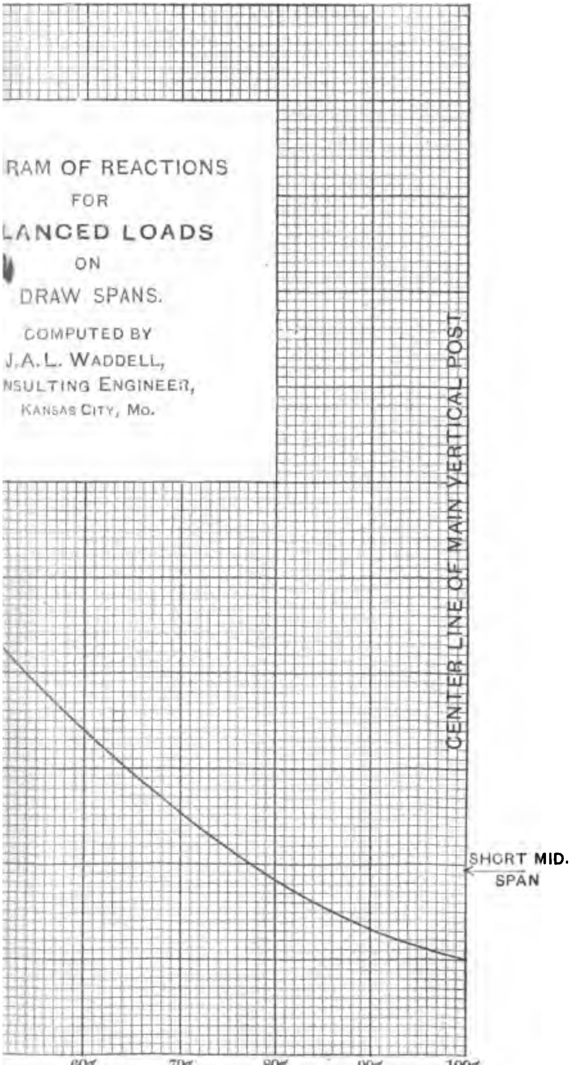


WIND LOADS FOR HI
J.A.L. WA
CONSULTING
POUNDS PER LIN. FT. OF SPAN.
1000
900
800
700
600
500
400
300
200
100
0
CLASS A.
" B.
" C.
CLASS A.
" B.
" C.
0
100
200
300
400
500
600
700
SPAN

PLATE VIII.


VAY BRIDGES.
EER,
ED CHORD.
ADED CHORD.
1000
900
800
700
600
500
400
300
200
100
0
POUNDS PER LIN. FT. OF SPAN.
800
900
1000
1100
1200
1300
1400
1500
FEET.


REACTIONS AT END OF SPAN.
100%
90%
80%
70%
60%
50%
40%
30%
20%
10%
0
END OF ARM
0
10%
20%
30%
40%
ONE ARM


RAM OF REACTIONS
FOR
LANCED LOADS
ON
DRAW SPANS.
COMPUTED BY
J. A. L. WADDELL,
NSULTING ENGINEER,
KANSAS CITY, MO.
CENTER LINE OF MAIN VERTICAL POST
SHORT MID. SPAN
60%
70%
80%
90%
100%

DRAW SPAN.

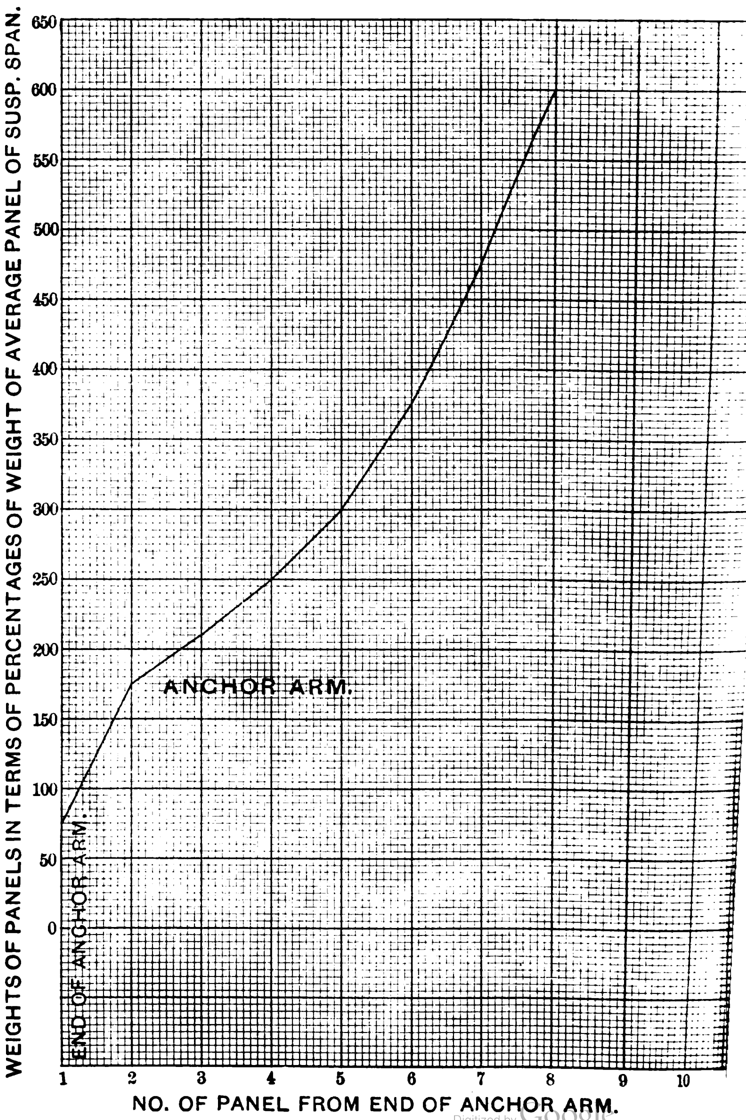
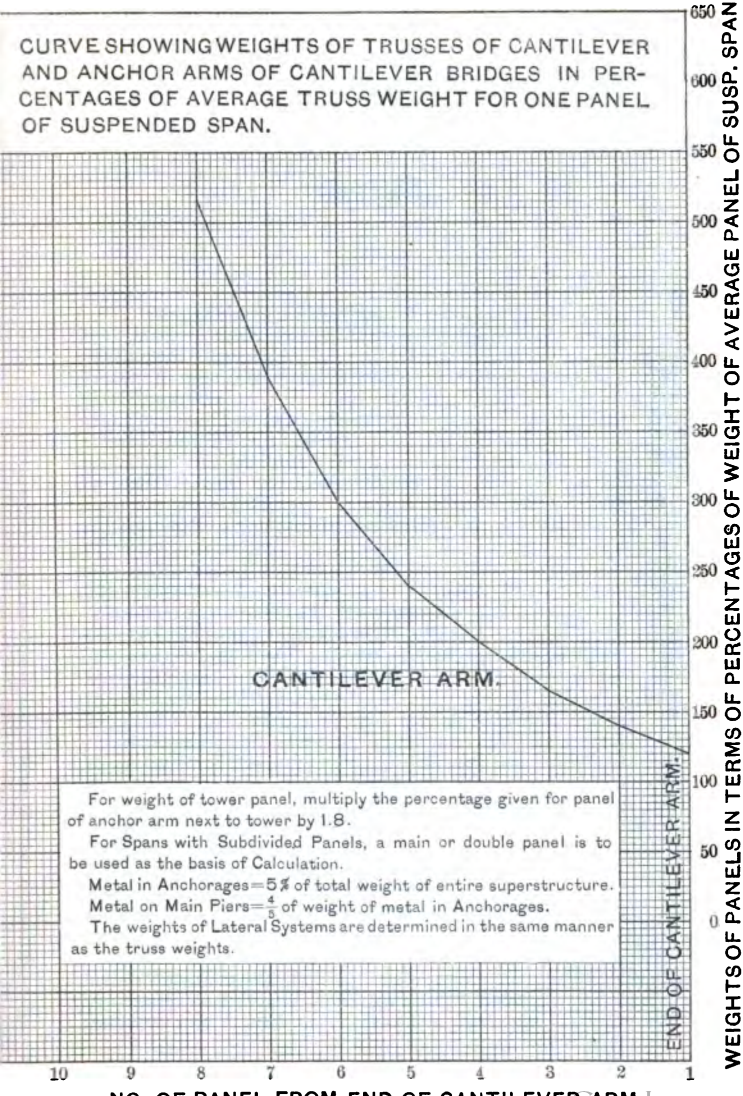

WEIGHTS OF PANELS IN TERMS OF PERCENTAGES OF WEIGHT OF AVERAGE PANEL OF SUSP. SPAN.
650
600
550
500
450
400
350
300
250
200
150
100
50
0
ANCHOR ARM.
END OF ANCHOR ARM.
1
2
3
4
5
6
7
8
9
10
NO. OF PANEL FROM END OF ANCHOR ARM.

PLATE X.


CURVE SHOWING WEIGHTS OF TRUSSES OF CANTILEVER AND ANCHOR ARMS OF CANTILEVER BRIDGES IN PERCENTAGES OF AVERAGE TRUSS WEIGHT FOR ONE PANEL OF SUSPENDED SPAN.
CANTILEVER ARM.
For weight of tower panel, multiply the percentage given for panel of anchor arm next to tower by 1.8.
For Spans with Subdivided Panels, a main or double panel is to be used as the basis of Calculation.
Metal in Anchorages=5% of total weight of entire superstructure.
Metal on Main Piers=4/5 of weight of metal in Anchorages.
The weights of Lateral Systems are determined in the same manner as the truss weights.
END OF CANTILEVER ARM.
650
600
550
500
450
400
350
300
250
200
150
100
50
0
10
9
8
7
6
5
4
3
2
1
WEIGHTS OF PANELS IN TERMS OF PERCENTAGES OF WEIGHT OF AVERAGE PANEL OF SUSP. SPAN.
NO. OF PANEL FROM END OF CANTILEVER ARM.

INDEX.

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